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Vol. 28 MARCH, 1936 No. 3

MODEL AND FULL SIZE SURGE TESTS ON LARGE BRANCHED TUNNEL SYSTEM OF DETROIT WATER SUPPLY

By Arthur C. Michael

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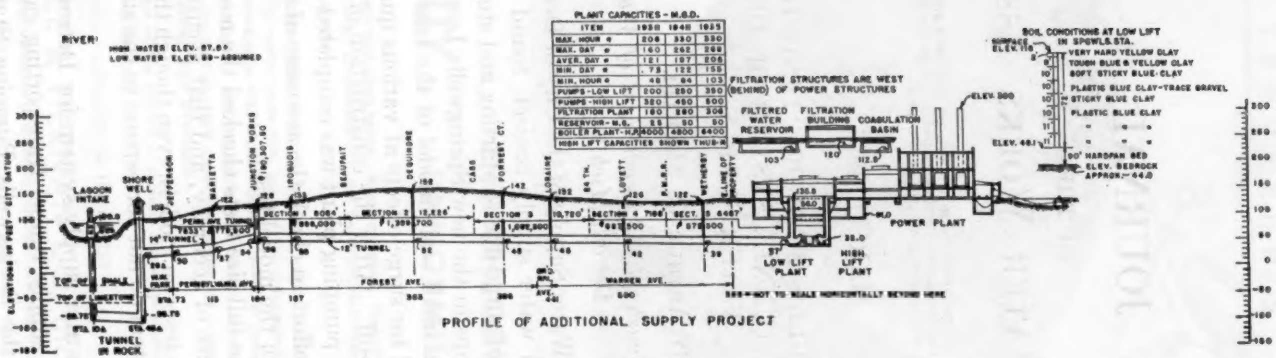
The Department of Water Supply of the City of Detroit, in designing its new additional water supply project, found it desirable to investigate the effects of surge due to starting and stopping of motor driven centrifugal pumps in the new Springwells Low Lift Pumping Plant. Accordingly, a small scale model of the tunnel system was constructed and tested for surge effects at various quantities of flow and conditions of shut-off. After the construction of the raw water supply tunnels and the pumping plant was completed full scale surge tests were made in an effort to check the accuracy of the predictions made from the results of the model tests.

It was found that the full size tests checked the model test results with a remarkable degree of accuracy, and that accurate predictions can be made from surge tests on models, even though the sizes of surge wells in the full size system do not correspond to the sizes used in the design of the model.

The new additional water supply system for Detroit consists of a long branched gravity supply tunnel transporting raw water from the Detroit River to the Springwells Pumping Station, which is



ITEM	1938	1948	1955
MAX. HOUR ◆	206	330	330
MAX. DAY ◆	160	262	280
AVER. DAY ◆	121	197	206
MIN. DAY ◆	.73	122	155
MIN. HOUR ◆	48	84	103
PUMPS - LOW LIFT	200	250	350
PUMPS - HIGH LIFT	320	450	500
FILTRATION PLANT	160	280	306
RESERVOIR - M.G.	25	50	50
BOILER PLANT - H.P.	4000	4800	6400
HIGH LIFT CAPACITIES SHOWN THUS ◆			

FIG. 1. PRELIMINARY DRAWING OF THE NEW TUNNEL SYSTEM AND PLANTS
The model was generally constructed from this design

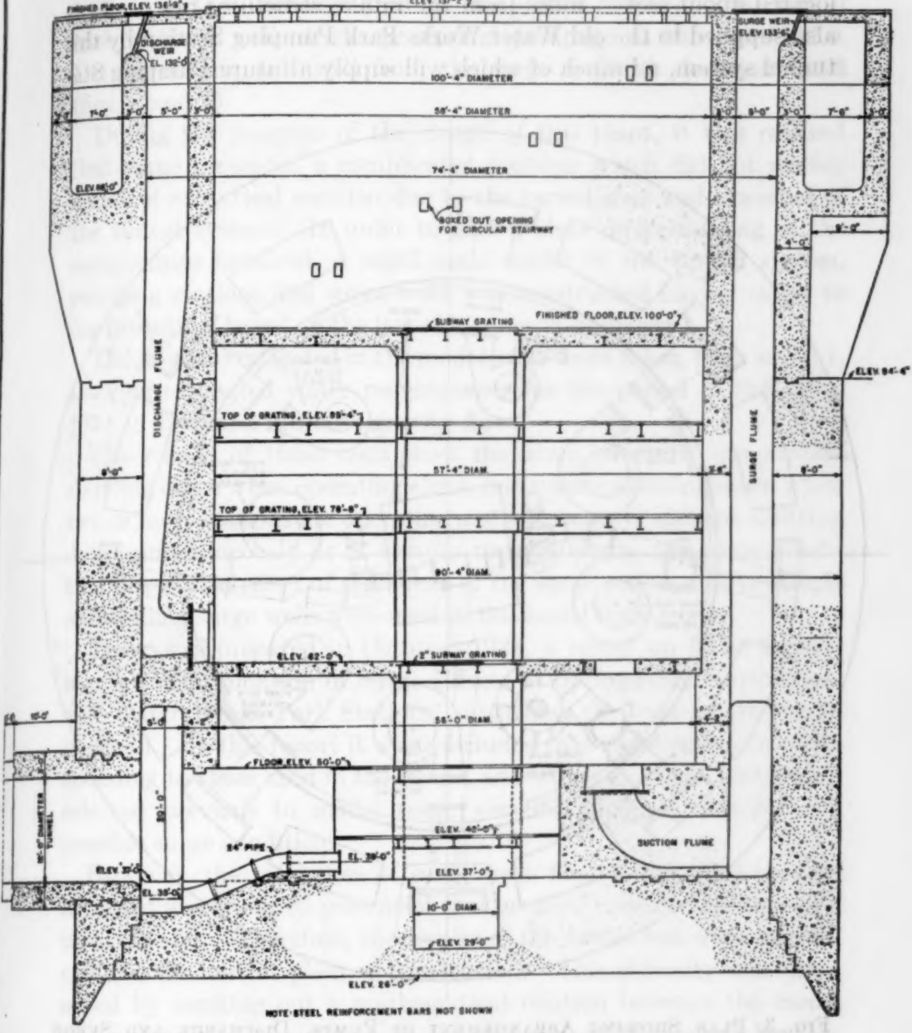


FIG. 2. VERTICAL SECTION THROUGH SPRINGWELLS LOW LIFT PLANT

On the right is shown the pump suction flume, which draws water from the annular suction tunnel, which is connected to the surge flume. Water which surges high enough spills over weir at elevation 133.5 into annular discharge flume. On the left is shown the pump discharge flume from which water spills over weir at elevation 132.0 into annular discharge flume.

Center line of pumps is at elevation 65.0 and base of motors at elevation 100.0.

located about eleven miles from the intake structure; raw water is also supplied to the old Water Works Park Pumping Station by this tunnel system, a branch of which will supply a future Pumping Sta-

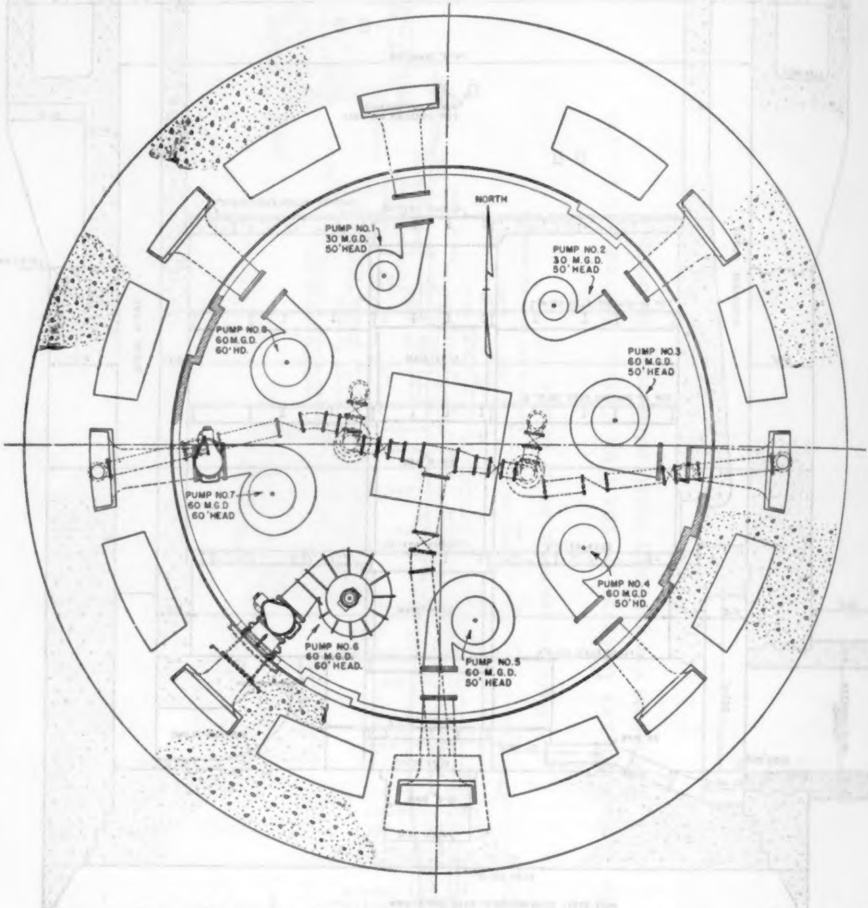


FIG. 3. PLAN SHOWING ARRANGEMENT OF PUMPS, DISCHARGE AND SURGE FLUMES

There are eight pump discharge flumes and eight surge flumes. The surge flumes have a total area of 446.4 square feet.

tion to be located in the North East section of the City. (See figure 1.) The Springwells low lift plant is so designed as to incorporate a series of simple surge wells within its walls, forming the suction chambers for the pumps. Alternate flumes between these same

walls form the discharge lines for the pumps. Weirs at the top of each of these wells discharge normal pumpage at elevation 132.0, Detroit datum, and spill from the surge wells at elevation 133.5 (figs. 2 and 3).

During the progress of the design of this plant, it was realized that surge presented a complicated problem which did not readily admit of analytical solution due to the varied sizes and branches of the tunnel system. In order to gain a clear understanding of the complexities involved, a small scale model of the tunnel system, pumping stations and surge wells was constructed having ratios to the prototype based on the law of kinematic similarity.

The flows investigated in the model tests were taken from table 1, showing estimated water requirements for the period in the years 1931 to 1950 and ultimate gravity flow.

The results of these tests show the surge effects of one station shutting down when operating alone, one station shutting down when two or more stations are operating and two or more stations shutting down simultaneously or at various time intervals, the latter condition causing an effect of resonance of the surge waves. Both simple and spilling surge wells were used in the model tests.

There was prepared in October, 1928, a report on "The Experimental Determination of Surge Effects at Springwells, North East, and Water Works Park Stations" describing the tests on the model in detail. In this report it was concluded that the surge wells corresponding to those used in the model, and which had been tentatively selected previous to model tests, were of adequate size for any possible surge condition.

However, the total area of surge wells finally built into the low lift plant was about 65 percent of that used for constructing the wells in the model. Therefore, the results of the model test were inapplicable to the low lift plant as constructed. This difficulty was eliminated by working out a mathematical relation between the model test results and the surges to be expected in the actual low lift plant, after which the model test results could be checked against actual service performance.

APPARATUS

The model, designed to simulate the tunnel system, consisted of three branches with surge wells, needle valves and quick closing valves at each of the three stations to represent low lift pumping stations.

TABLE 1

Estimated flows and losses of head in tunnel system based on estimated future water consumption

Flows in the model were based on figures in this table

ITEM	1931			1940			1950			ULTIMATE	
	Minimum day	Average day	Maximum day	Minimum day	Average day	Maximum day	Minimum day	Average day	Maximum day	Gravity flow	25-foot boost
Springwells Station Only:											
Elevation of water in river, feet	97	93	90	97	93	90	97	93	90		
Delivery to Springwells, m.g.d.	91	121	164	147	197	266	155	206	278		
Loss of head to Springwells, feet	2.0	3.3	6.0	4.9	8.4	14.7	5.4	9.1	16.0		
Loss of head to Junction Well, feet	0.33	0.56	0.99	0.81	1.41	2.5	0.89	1.5	2.7		
Loss of head to Shore Well, feet	0.08	0.13	0.23	0.34	0.34	0.6	0.21	0.3	0.6		
Loss of head to auxiliary low lift, feet			0.29			0.7			0.8		
Elevation of water at Springwells, feet	95.0	89.7	84.0	92.1	84.6	75.3	91.6	83.9	74.0		
Springwells and Water Works Park Station:											
Elevation of water in river, feet	97	93	90	97	93	90	97	93	90	90	
Delivery to Springwells, m.g.d.	91	121	164	147	197	266	155	206	278	397	
Delivery to Water Works Park, m.g.d.	123	164	221	166	221	298	173	230	310	370	
Loss of head to Springwells, feet	2.4	3.8	7.1	5.6	9.8	16.9	6.3	10.5	18.5	35.5	
Loss of head to Junction Well, feet	0.7	1.2	2.2	1.6	2.8	4.9	1.7	3.0	5.3	9.4	
Loss of head to Shore Well, feet	0.4	0.6	1.2	0.8	1.5	2.5	0.9	1.6	2.7	4.5	
Loss of head to auxiliary low lift, feet			1.9			3.9			4.3	6.8	
Elevation of water at Springwells, feet	94.6	89.2	82.9	91.4	83.2	73.1	90.7	82.5	71.5	54.5	

Springwells and North East Station:										
Elevation of water in river, feet				97	93	90	97	93	90	90	90
Delivery to Springwells, m.g.d				114	152	204	155	206	278	374	488
Delivery to North East Station, m.g.d				59	79	107	95	126	170	243	347
Loss of head to Springwells, feet				3.7	6.2	11.0	6.7	11.4	19.9	35.5	60.5
Loss of head to North East Station, feet				2.3	3.8	7.0	5.1	8.5	15.0	29.0	54.0
Loss of head to Junction Well, feet				1.1	1.8	3.4	2.2	3.8	6.6	12.4	21.9
Loss of head to Shore Well, feet				0.2	0.5	0.8	0.5	0.9	1.6	3.0	5.4
Loss of head to auxiliary low lift, feet						1.0			2.0	3.7	
Elevation of water at Springwells, feet				93.3	86.8	79.0	90.3	81.6	70.1	54.5	54.5
Springwells, Water Works Park, and North East Station:											
Elevation of water in river, feet				97	93	90	97	93	90	90	90
Delivery to Springwells, m.g.d				114	152	204	155	206	278	346	462
Delivery to Water Works Park, m.g.d				140	187	252	173	230	310	370	370
Delivery to North East Station, m.g.d				59	79	107	95	126	170	217	321
Loss of head to Springwells, feet				4.4	7.3	13.1	7.9	13.5	23.4	35.5	60.5
Loss of head to North East Station, feet				3.0	4.9	9.1	6.3	10.6	18.5	29.0	54.0
Loss of head to Junction Well, feet				1.8	2.9	5.5	3.4	5.8	10.0	15.5	26.2
Loss of head to Shore Well, feet				0.8	1.5	2.5	1.4	2.5	4.4	6.7	10.1
Loss of head to auxiliary low lift, feet						3.7			6.4	9.5	
Elevation of water at Springwells, feet				92.6	85.7	76.9	89.1	79.5	66.6	54.5	54.5

The scales of the model were determined by the law of kinematic similitude, which has been used extensively in various hydraulic investigations. The reader is referred to an article written by Professor W. F. Durand of Stanford University, California, entitled "Application of the Law of Kinematic Similitude to the Surge Tank Problem."[1]

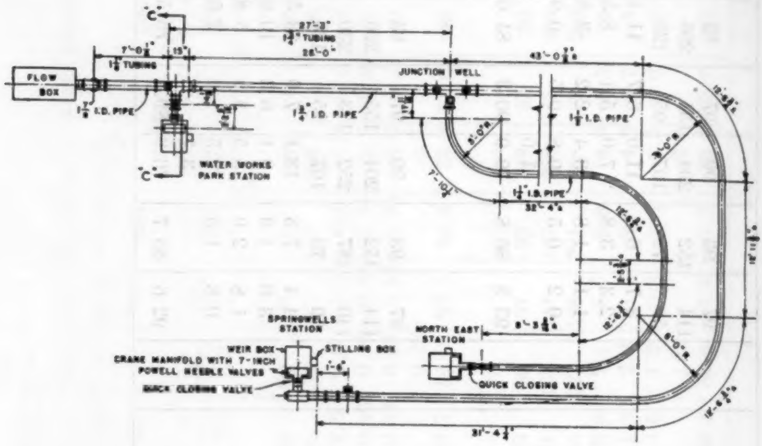


FIG. 4. PLAN OF SURGE MODEL

Circles shown immediately upstream from quick closing valves are surge wells.

The scales or ratios of prototype to model were as follows:

Velocity ratio = 3.
Length ratio = 400.
Diameter ratio = 96.
Springwells section (12 feet 0 inch tunnel) height ratio = 20.96.
Springwells section time ratio = 57.24.
Volume of spill ratio (Springwells section) = cubic centimeters × 418.07 = gallons (spill only).
Quantity ratio = (diameter ratio)^2 × velocity ratio.
Surge well diameter ratio (Springwells) = 274.77.

The 15.5 foot diameter river tunnel was represented by $1\frac{7}{8}$-inch I. D. drawn copper tubing; the 11,000 feet of 14-foot diameter tunnel

[1] Proceedings of The American Society of Mechanical Engineers, October, 1921.
[2] Canadian Engineer, Vol. 27, August 20, 1914.

was represented by 27.25 feet of $1\frac{3}{4}$-inch tubing; the approximately 45,000 feet of 12-foot diameter Springwells tunnel was represented by 112.5 feet of $1\frac{1}{2}$-inch tubing; and 31,000 feet of 10-foot diameter tunnel was represented by 77.5 feet of $1\frac{1}{4}$-inch copper tubing. Space limitation required long radius bends in some of the tubing; this bending was very carefully done after which a hardwood ball having a diameter a few thousandths of an inch less than that of the tubing before bending was required to be passed without binding. Pictures and drawings show details of the model (figs. 4, 5, 6, 7 and 8).

MODEL TEST PROCEDURE

Previously calibrated V-notch weir boxes were used to measure the flow of water at each miniature pumping station, the level over

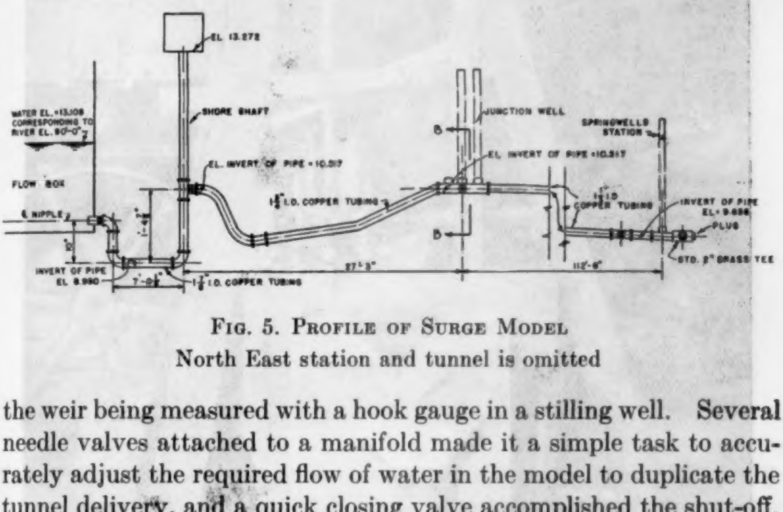


FIG. 5. PROFILE OF SURGE MODEL
North East station and tunnel is omitted

the weir being measured with a hook gauge in a stilling well. Several needle valves attached to a manifold made it a simple task to accurately adjust the required flow of water in the model to duplicate the tunnel delivery, and a quick closing valve accomplished the shut-off.

A graduated glass tube of uniform diameter was placed in the pipe line near the inlet of the quick closing valve to represent the surge well. A flow box in which a constant water level was maintained supplied water to the system and represented the river supply.

After the model was set up, it was tested to determine the values of C and n in the formula $H = CV^n$, where H = loss of head, V = mean velocity, and C and n are respectively coefficient and exponent which remain constant for any given system. After this determination of values peculiar to the model it was possible to compute the values of all scale relations not originally assumed, since all values

pertaining to the tunnel were assumed or computed in the progress of its design.

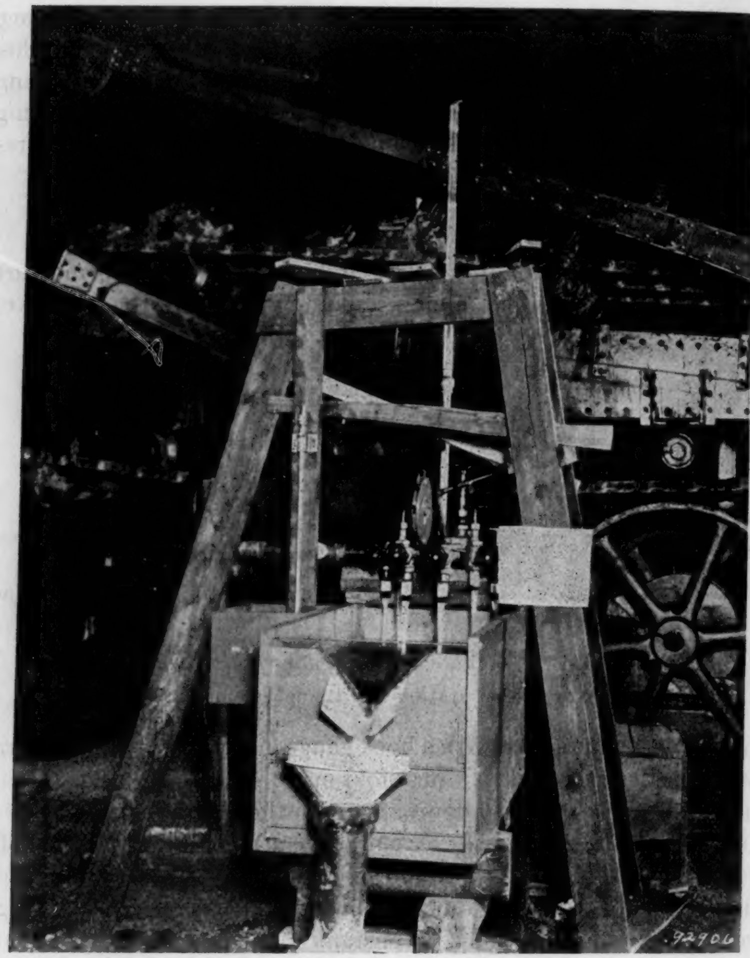


FIG. 6. PHOTOGRAPH OF ONE OF THE V-NOTCH WEIR BOXES IN OPERATION SHOWING HOOK GAUGE SET IN STILLING WELL, REGULATING NEEDLE VALVES, QUICK CLOSING VALVES AND SURGE WELL

The test procedure from this point was routine, and consisted of measuring surges resulting from sudden shut down of stations at various flows. The surge cycle in such a relatively small model occurs

quite rapidly, an alert observer being required to read the maximum and minimum points on the surge curve. The vertical distance between the water level in the surge tube just prior to shut down and the first maximum point of surge rise (surge increment) was then multiplied by the height ratio for that particular section of tunnel, (no two sections had the same height ratio), the result being the surge increment to be expected in the full size surge well. This increment,

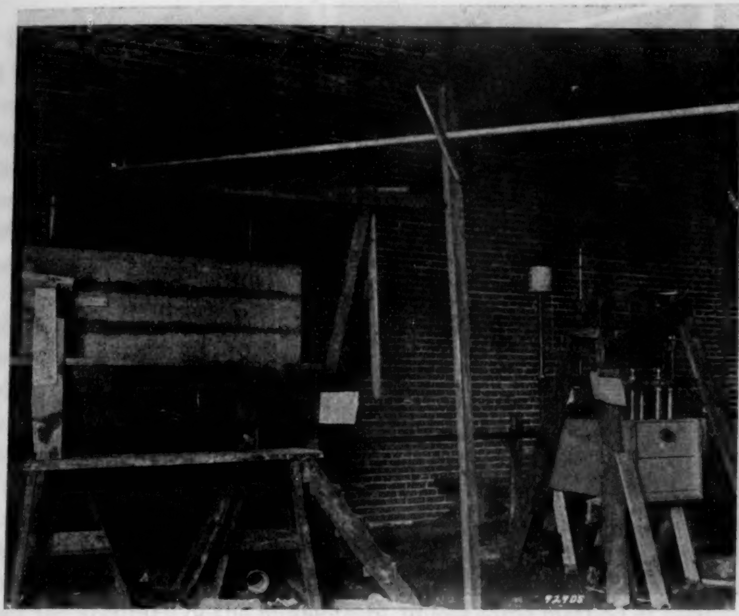


FIG. 7. PHOTOGRAPH OF THE FLOW BOX, WHICH REPRESENTED THE RIVER AND SUPPLIED ALL THE WATER TO THE MODEL AT A CONSTANT HEAD

On the right is the miniature of the Water Works Park Plant. The can at the top of the vertical pipe represents the surge basin for the shore well.

when added to the computed elevation of the hydraulic gradient in the surge well just prior to a power failure or pump shut down resulted in the elevation to which the water would be expected to rise in the full scale surge well for a flow corresponding to that used in the model tunnel.

All this work required a great deal of care to insure accuracy due to the large multiplying factors involved. Air released from the water and trapped in the tubing was cleaned out frequently and all

hook gauge settings were checked daily. The report on the model test stated that

The accuracy of the final result of this experiment is limited by the following factors:

1. Inaccuracy of setting and reading of the hook gauges. A difference of one one-thousandth of a foot on the hook gauge corresponds to about 11,000,000 gallons daily in the tunnel system.

Fig. 8. Photograph Showing a General View of the Model

In left foreground is the Water Works Park Station and in right background are the Springwells and North East Stations.

2. Varying river elevations. The river elevation used in the test was taken as 90.0, but the river has been known to stand as high as 97.0.

3. Inaccuracy of computed loss of head in the tunnel system. This was computed by the Hazen and Williams formula using C as 120.

The actual value of C for the new tunnel has been checked and found to be approximately 135.0.

FIELD TESTS

After the Springwells Low Lift Pumping Plant was completed and placed in operation surges resulting from 2, 3 and 4 pumps being

shut down simultaneously were measured. No attempt has been made to produce resonance effects nor to build up a maximum surge at any one point, as was done in the model test. All shut-offs were made at the new Springwells Station. Further, no attempt has been made to check the model test results at points other than the Springwells Low Lift Plant, because of the relative unimportance of surge at these points.

Procedure

Observers, equipped with synchronized watches, were stationed at the various surge wells along the tunnel route. A predetermined flow was established in the Springwells section of tunnel by starting the necessary pumps in the Springwells Low Lift Plant; this flow was maintained for a period of thirty minutes to an hour in order to dissipate any surge energy caused by drawdown. At a prearranged time all pumps in the Springwells Low Lift Station were shut down simultaneously. Readings were taken of the water surface elevation at thirty second to one minute intervals from about ten minutes before closure through about three complete surge cycles. In the case of spill over the spilling weir, the time of spill was recorded as well as readings of the height of water over the weir crest at fifteen second intervals. The weir coefficient, which had previously been determined, was then used to compute the quantity of total spill.

The first or maximum surge increment was of greatest importance in this work. As had been mentioned before, the field tests were not comparable to the model tests, because of a difference in the corresponding surge well areas. This condition made it impossible to check the accuracy of the model tests, unless some method of relating the surge in the tunnel system to the surge corresponding to that in the model could be found.

Mathematical formulae were finally derived to set up this relationship, after which it became possible to check the model tests against the measurements made on the completed system.

DERIVATION OF FORMULAE

The equivalent area of surge well used in the model for the Springwells Low Lift Plant was 675 square feet, and the total surge well area actually built into the plant was 446.4 square feet or a ratio of prototype to model of about 66 percent. This meant, of course, that the actual surge effects would be greater than those determined by the model tests. This difference in surge well areas was the only consideration required in the development of the formulae.

The derivation of these formulae depends upon the theory of work when the amount of energy available is balanced against the amount of work done. The general method is based on the translated article of Prof. Franz Prazil "Surge Tank Problems."[2]

The available energy consists of the kinetic energy of the water in motion in the tunnel system and the potential energy of the amount of water admitted to the surge well dropping through a distance equal to the total friction head. This available energy is dissipated in two ways; first, by lifting the water in the surge well, and second, by friction work. Thus the energy balance equation may be stated as:

$$K + P = Fw + Lw \tag{1}$$

where K is equal to kinetic energy, P is equal to potential energy, Fw is equal to friction work, and Lw is equal to lifting work.

There are three conditions of relation between surge in model and surge in tunnel system!

First, where a simple surge rise occurs in both systems. No spill.

Second, where spill occurs in both systems.

Third, where a simple surge rise occurs in the model and a spill occurs in the low lift plant.

Formula for first condition:

$$H = 0.3F \pm \sqrt{1.51\, H_1(H_1 - 0.6F) + 0.09F^2} \tag{I}$$

Formula for second condition:

$$Q = \frac{(y - 0.3F)\, Q_1 + 119.8y^2 - 71.88Fy}{(y - 0.3F)} \tag{II}$$

Formula for third condition:

$$Q = \frac{1.665H_1^2 - H_1F - 1.1y^2 + 0.66Fy}{0.00494\,(y - 0.3F)} \tag{III}$$

To derive equation (I) assume two tunnel systems exactly alike except that each has a different surge tank area. For the same flow and shut-off conditions the surge in each tank will be different from that of the other and will be greater for the tank having the smaller area.

The following system of notation applies to that which follows:

q = Flow in tunnel in c.f.s.

L = Length of tunnel in feet.

a = Area of tunnel.

V = Velocity of water in tunnel just prior to shut off.
A = Area of low lift plant surge well.
A_1 = Area corresponding to model surge well.
F = Total loss of head from river to low lift plant (this is the same for both systems).
H = Height of maximum surge rise in low lift plant surge well corresponding to area A (surge increment).
H_1 = Height of maximum surge rise in surge well equivalent to well in model, corresponding with well of area A_1 (surge increment).

Note: H and H_1 will be called surge increments and represent the vertical distance between the elevation of the hydraulic gradient just prior to shut-down and the elevation to which the first (or maximum) surge cycle rises.

K = Kinetic energy in the tunnel due to flow q.
P = Potential energy of the amount of water admitted to the surge well.
Fw and Fw' = Friction work done in dissipating part of the available energy in the full size system and the model equivalent respectively.
Q = Total quantity of water spilled in cubic feet.
h = The friction work done in the whole system during the first half of surge cycle, divided by the weight of water admitted to the surge well above the hydraulic gradient.
$h = \dfrac{Fw}{62.4\,AH}$ for the low lift plant.
y = The distance from the hydraulic gradient in the surge well prior to shut down, to the center of gravity of the water above the weir crest during spill. This distance varies with the height of water over the weir but the percentage of variation to the total distance is so small that y is considered constant.
Lw = Lifting work done in raising water in the surge well.

The kinetic energy in the tunnel system is equal to

$$K = \frac{MV^2}{2} = \frac{62.4\,aLV^2}{32.2 \times 2} = 0.97\,aLV^2 \qquad (2)$$

This remains the same for both systems.

Weight of water admitted to surge well:

$$\begin{aligned} &1.\ \text{For low lift plant} &= 62.4\,AH \qquad (3)\\ &2.\ \text{For model equivalent} &= 62.4\,A_1H_1 \qquad (4)\end{aligned}$$

Potential energy of this weight of water falling through the distance F:

$$\begin{aligned} &1.\ \text{For low lift plant} &= P = 62.4\,AHF \qquad (5)\\ &2.\ \text{For model equivalent} &= P' = 62.4\,A_1H_1F \qquad (6)\end{aligned}$$

Lifting work done in raising a column of water in the surge well:

$$1.\ \text{For low lift plant} = Lw = 62.4\,AH\frac{H}{2} = 31.2\,AH^2 \qquad (7)$$

$$2.\ \text{For model equivalent} = Lw' = 31.2\,A_1H_1^2 \qquad (8)$$

From the basic equation (1) we have

$$K = Fw + Lw - P$$

Substituting the values of Lw and P (equations (5) and (7)) we have:

$$K = 31.2\,AH^2 + Fw - 62.4\,AHF \text{ for low lift plant}$$

and similarly

$$K = 31.2\,A_1H_1^2 + Fw' - 62.4 - A_1H_1F \text{ for the model equivalent.}$$

Since K is the same for both systems we may equate these two values for K and we have,

$$31.2\,AH^2 + Fw - 62.4\,AHF = 31.2\,A_1H_1^2 + Fw' - 62.4\,A_1H_1F \qquad (9)$$

Now, since

$$h = \frac{Fw}{\text{Weight of water admitted to surge well}}$$

and

$$Fw = (\text{weight of water admitted to surge well})\ h$$

Note (this is the method suggested by Professor Prazil):
From (3) and (4)

$$\begin{aligned} Fw &= 62.4\ A\ H\ h \text{ for low lift plant} \qquad (10)\\ Fw' &= 62.4\ A_1H_1h \text{ for model equivalent} \qquad (11)\end{aligned}$$

Substituting values in equations (10) and (11) in equation (9) we have:

$$31.2\ AH^2 + 62.4\ AHh - 62.4\ AHF = 31.2\ A_1H_1^2 + 62.4\ A_1H_1h - 62.4\ A_1H_1F \qquad (12)$$

Dividing through by 31.2 A we have

$$H^2 + 2hH - 2FH = \frac{A_1}{A}H_1^2 + \frac{2A_1}{A}hH_1 - \frac{2A_1}{A}FH_1$$

or

$$H^2 + 2(h - F)H = \frac{A_1}{A}H_1(H_1 + 2h - 2F) \qquad (13)$$

Completing the square by adding $(h - F)^2$ to both sides of equation (13) there results:

$$H^2 + 2(h - F)H + (h - F)^2 = \frac{A_1}{A}H_1(H_1 + 2h - 2F) + (h - F)^2$$

$$H + (h - F) = \pm\sqrt{\frac{A_1}{A}H_1(H_1 + 2h - 2F) + (h - F)^2}. \qquad (14)$$

We know that

$$\frac{A_1}{A} = \frac{675.0}{446.4} = 1.51$$

Therefore

$$H = -(h - F) \pm \sqrt{1.51\ H_1(H_1 + 2h - 2F) + (h - F)^2} \qquad (15)$$

Professor Prazil, on page 455, "Canadian Engineer," September 17, 1914, concludes that an average value of 0.7 F should be taken for h; this assumption is made here. Our formula thus becomes:

$$H = 0.3F \pm \sqrt{1.51\ H_1(H_1 - 0.6F) + 0.09F^2} \qquad \text{(I)}$$

The value of the surge increment H_1, which was predicted from the model tests, is known; the friction head F is known or can be computed for any quantity of flow, and therefore it is possible to solve equation (I) for the value of H, the surge increment to be expected in the completed low lift plant surge well. Equations II and III may be similarly derived.

These formulae were all checked experimentally and found to be

correct within a reasonable percentage, except for formula II, which relates spill in the model to spill in the completed system. Formula II results in values of Q which are too low because the spilling weir used in the model was restricted and thus reduced the quantity Q. The spilling weir in the low lift plant is not restricted.

The predictions made from the model tests were first corrected for river elevation and loss of head, and then transposed by use of the formulae as derived above. These results checked the values ob-

TABLE 2

Data on tests made on full size tunnel system

Figure 12 shows data omitted under column headed "Predicted Surge Increment."

This table includes data shown more clearly in figure 12, in graphic form.

TEST NUMBER	DATE	RIVER ELEVATION	FLOW TO WATER WORKS PARK	FLOW TO SPRINGWELLS	ELEVATION, WATER SURFACE AT SPRINGWELLS	SPRINGWELLS LOW LIFT PUMPING STATION						REMARKS
						Surge increment	Predicted surge increment	Maximum elevation, surge rise	Duration of spill	Amount of spill	Predicted spill	
			m.g.d.	*m.g.d.*	*feet*	*feet*			*seconds*	*gallons*	*gallons*	
1	12/12/32	91.9	180	130	89.35	34.95	See remarks	124.3				No flows in model as low as 130 m.g.d. (see fig. 12)
2	2/ 3/33	92.75	180	153	88.01	42.09	See remarks	130.1				Predicted surge increment for 164 m.g.d. was 44.3
3	5/ 3/33	94.12	170	197	87.11				58	26,200	25,800	
4	5/10/33	94.06	180	256	82.53				100	86,500	77,200	

tained in the field tests very closely, as can be seen from table 2 and figures 9, 10, 11, and 12.

It is thought that the discrepancies shown are due more to the inaccurate method of measuring spill in the low lift plant than to the inaccuracy of the model predictions. Simple surges check very well. The spill in the field tests was measured by calibrating one pump discharge weir with a venturi meter and assuming that the surge weirs had the same coefficient. It is highly probable that these spill figures are not very dependable, but they certainly show the

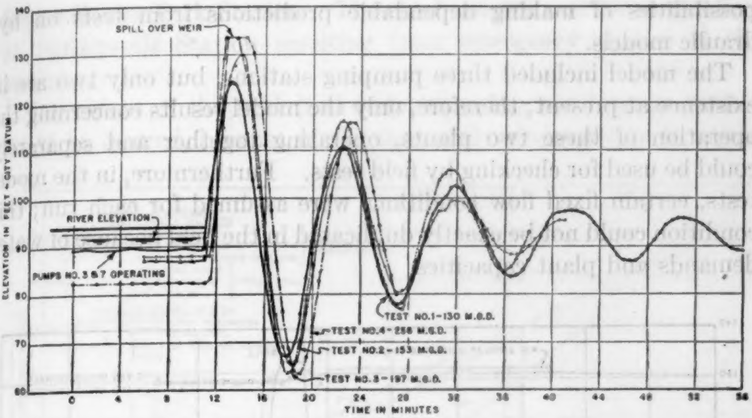


FIG. 9. WATER ELEVATIONS IN THE SPRINGWELLS LOW LIFT PLANT SURGE WELLS FOR ALL OF THE FIELD TESTS

Note the horizontal line denoting time of spill

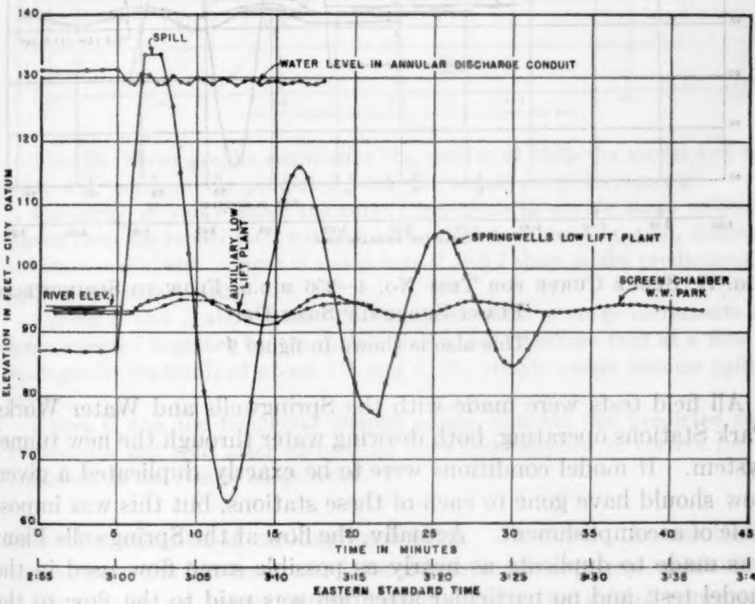


FIG. 10. SURGE CURVE FOR TEST NO. 3

This curve is also shown in figure 9, but is here plotted alone for ease of inspection.

possibilities of making dependable predictions from tests on hydraulic models.

The model included three pumping stations, but only two are in existence at present; therefore, only the model results concerning the operation of these two plants, operating together and separately could be used for checking by field tests. Furthermore, in the model tests, certain fixed flow conditions were assumed for each run; this condition could not be exactly duplicated in the field because of water demands and plant capacities.

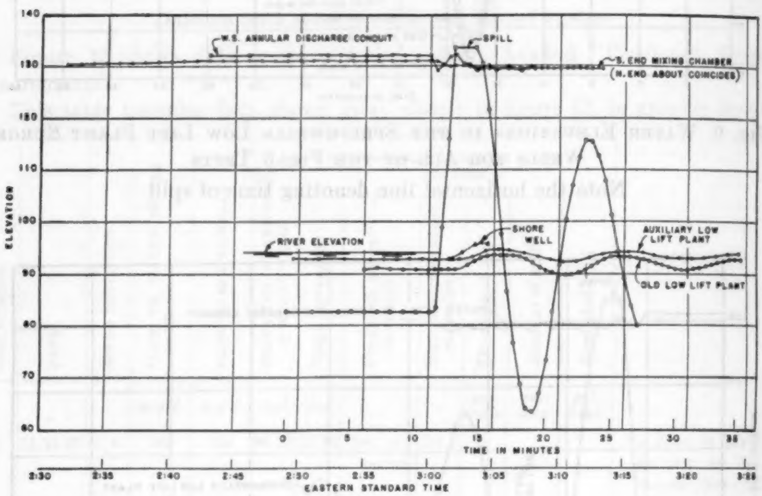


FIG. 11. SURGE CURVE FOR TEST NO. 4—256 M.G.D. FLOW TO SPRINGWELLS PLANT SUDDENLY SHUT OFF

This also is shown in figure 9

All field tests were made with the Springwells and Water Works Park Stations operating, both drawing water through the new tunnel system. If model conditions were to be exactly duplicated a given flow should have gone to each of these stations, but this was impossible of accomplishment. Actually, the flow at the Springwells Plant was made to duplicate as nearly as possible same flow used in the model test, and no particular attention was paid to the flow to the Water Works Park Station. It is thought that this procedure was justified because of the great distance between the two stations (about 11 miles), and because of the numerous surge wells near the

Water Works Park Station. The chief concern was for surge effects at Springwells Station resulting from emergency closure of that station.

The extent of the field (full scale) tests has not been as great as could be desired, because of the time and inconvenience to regular

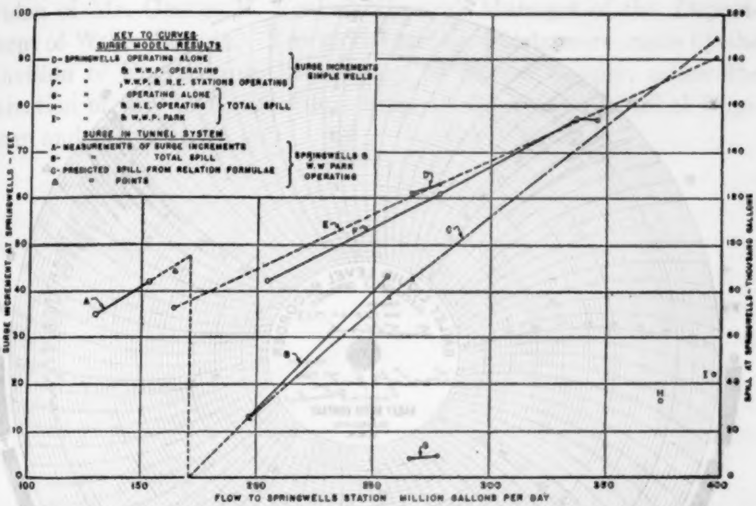


FIG. 12. These graphs summarize the results of both the model and field tests and compare the predicted with the actual surge increment.

Curves *D, E* and *F* show the surge increments in simple surge wells predicted from the model tests without corrections for river elevation, difference in size of wells, etc. Curve *G* and points *H* and *I* show model predictions for spill on the same basis.

Curves *A* and *B* show actual field measurements of surge increments and are connected together by the dotted line which indicates that at a flow (to Springwells Station), of about 171 m.g.d. the simple surges become spilling surges.

Curve *C* and the triangular shaped points indicate the predicted surge increments and spill after corrections have been applied to correct for the difference in the size of surge wells.

operation entailed in making such tests. However, the results obtained from the few tests which have been made are very satisfactory.

Practically no investigation was made on the model concerning drawdown due to starting of pumps. Generally, the drawdown surge has about the same initial intensity as the shut-off surge, but does not last as long because the pump consumes the excess energy rapidly

after it is started. Figure 13 is a water level chart which shows the drawdown caused by starting one 60 m.g.d. pump and also shows the surge effect of stopping this same pump; this is a very interesting chart in that it shows that the surge lasted more than six hours.

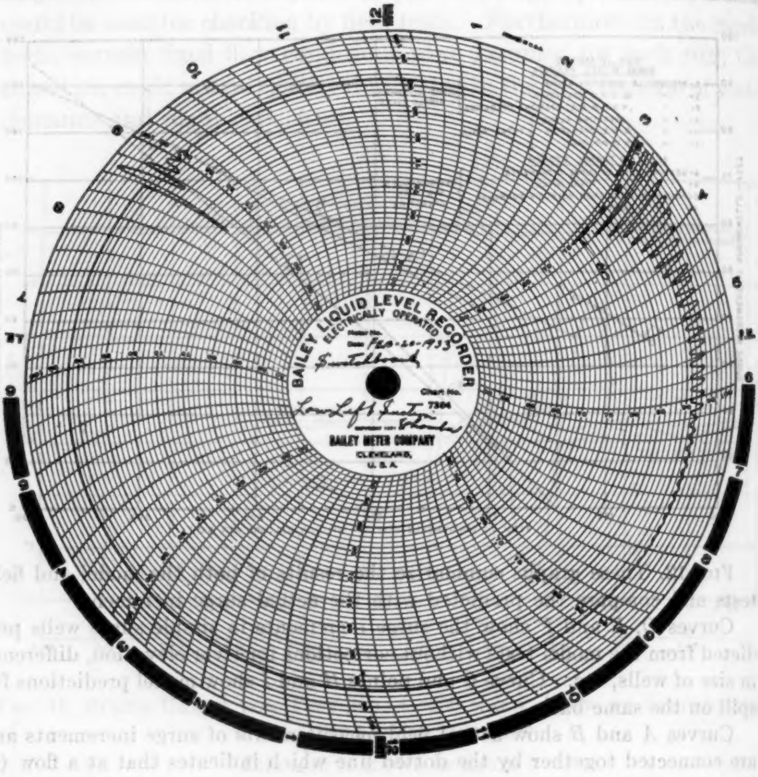


FIG. 13. PHOTOSTAT OF A CHART FROM AN ELECTRICAL WATER LEVEL RECORDING INSTRUMENT WHICH RECORDS THE WATER LEVEL ELEVATIONS IN THE SURGE WELLS OF THE SPRINGWELLS LOW LIFT PLANT

It shows the effect of starting a pump of 60,000,000 gallon daily capacity at 8:30 A.M. and of stopping the same pump slightly before 3:00 P.M. It is especially interesting in that it shows that the surge wave persists in the system for about 6 hours.

Neither was any study made of the effect of change of load. However, it has been noted from the surge well water level recording charts that about the same initial surge rise results from the sudden shut-down of one pump while others are operating, as if no other pump

were operating, the only difference being that in the former case the surge oscillation does not last as long because the excess energy is dissipated by the pumps which remain in service.

The design, construction and experimental work in connection with the new additional water supply system was under the general supervision of Mr. George H. Fenkell, General Manager of the Department of Water Supply. The model and field tests were made by the Division of Engineering, Department of Water Supply, under the direction of Mr. L. E. St. John, formerly Assistant Electrical Engineer, and the writer.

AIR CONDITIONING IN RELATION TO WATER CONSUMPTION

L. Logan Lewis

(Chief Engineer, Carrier Engineering Corporation, Newark, N. J.)

While it may be possible that air conditioning has but recently become of interest to you, it is by no means a novelty. The industry is some thirty years old. During the first twenty years of its life, application was limited almost entirely to industrial plants and, hence, few people knew of it. About ten years ago its application began to spread rapidly to theatres, department stores, restaurants and other buildings in which the public gather.

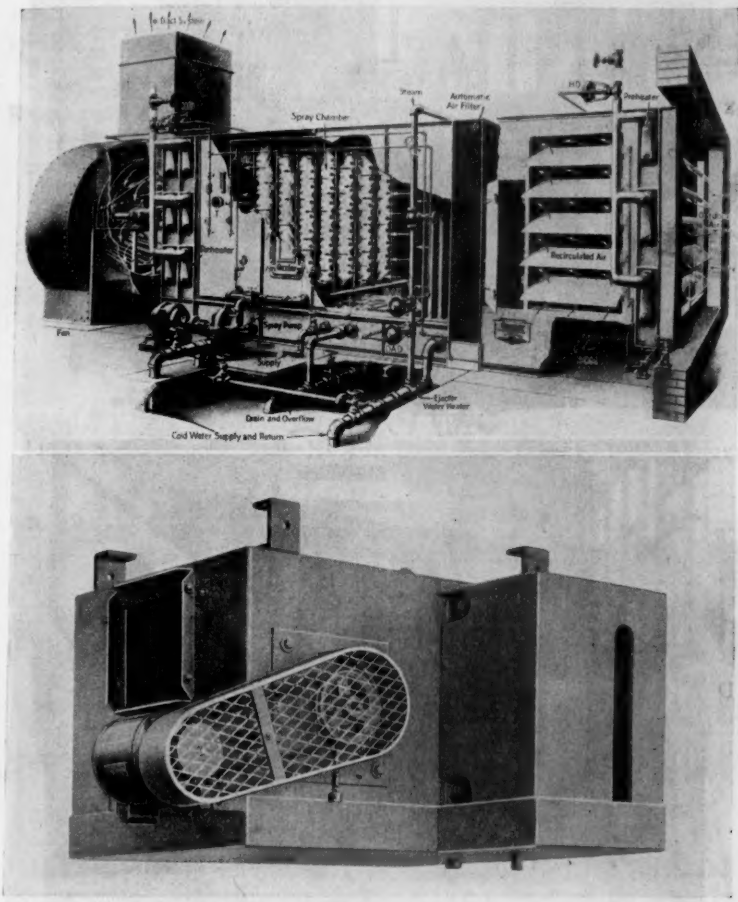
This second phase of development brought improvements in equipment, made price reductions possible, and greatly broadened the market.

For example, only a few years ago, Jones would have paid $10,000.00 for air conditioning his corner drug store, and would have been compelled to hire two licensed refrigerating engineers to operate his plant. Today, Jones can buy better equipment for less than $3,000.00 and can operate it himself with push buttons or thermostatic controls, placed in any convenient location in his store. It has now become a money-maker for Jones.

From an air conditioning point of view, every human being in Jones' store is a heater, giving off heat in both its sensible and latent form, or, with freedom of description, as dry and moist heat. Every light, every electrical device, every coffee urn is a heater. The rays of the sun heat the building structure where they strike it, and heat also flows in through it, because a lower temperature is maintained inside the structure. Therefore, his system must remove heat and moisture when the weather is warm and add heat and moisture when the weather is cold.

For effectively removing heat, he must have a refrigerating machine of which there are many types, all of which must be provided with means for removing the effluent heat. For example, a very elementary definition of a compression machine resolves it into three parts:

The cooler, which receives heat at a low temperature or head, the compressor, which raises the pressure of the refrigerant so that the heat received by the cooler can be removed at a relatively high temperature or head, and the condenser from which the heat is finally



A

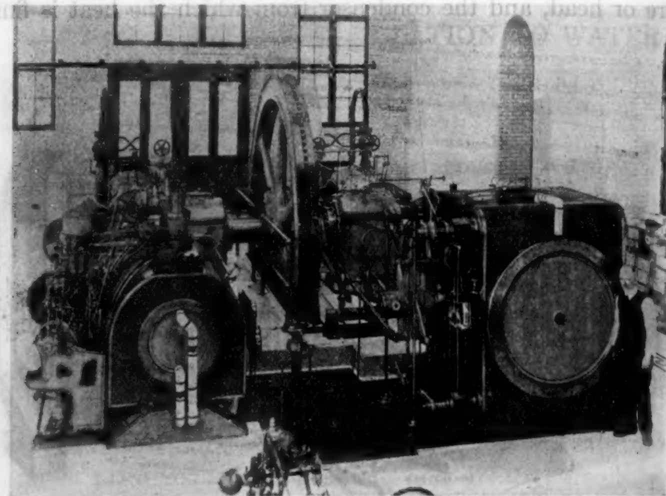
B

FIG. 1. THE OLD AND THE NEW IN AIR CONDITIONING EQUIPMENT

removed. This effluent heat usually is carried away wholly or partly by the water you furnish, and requirements may be great. In winter, when heat is added, Jones' requirements for water are minute.

Thus, his demand for water is a seasonal one and is proportional to the quantity of heat which must be removed.

C



D

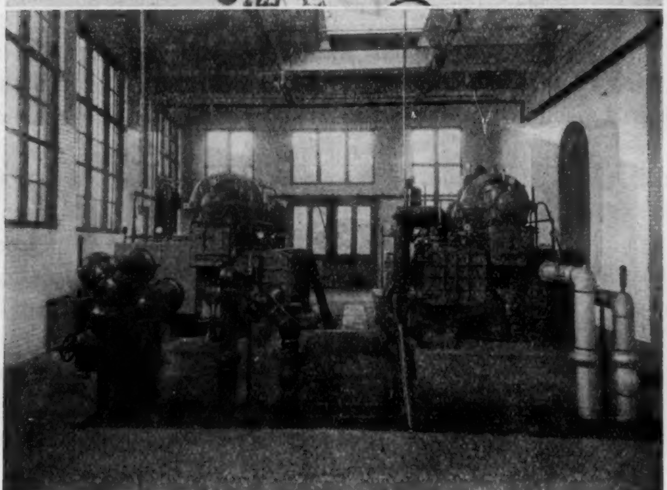


FIG. 2. THE OLD AND THE NEW IN LARGE REFRIGERATING EQUIPMENT

A simple way for him to get rid of this heat is to buy water from you and pass it through his condenser to the sewer. If this were the only way for him to do it, you might well be concerned by the

E F

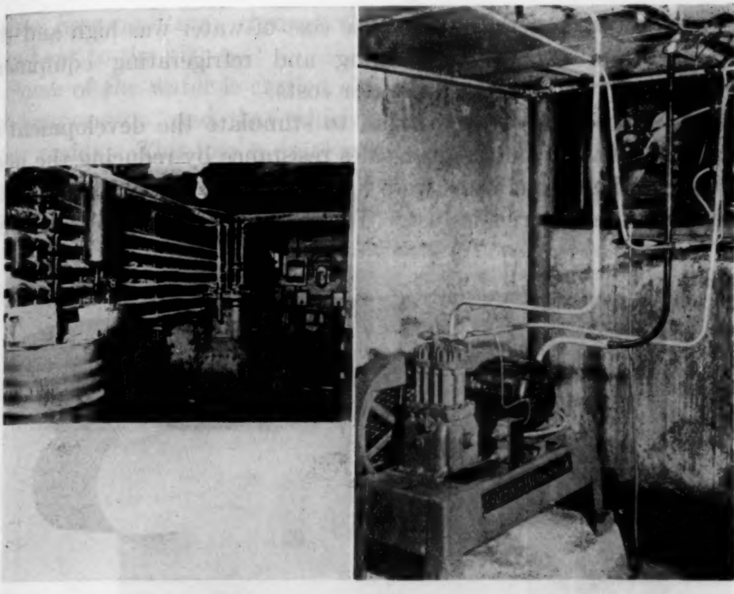


FIG. 3. THE OLD AND THE NEW IN SMALL REFRIGERATING EQUIPMENT

G H

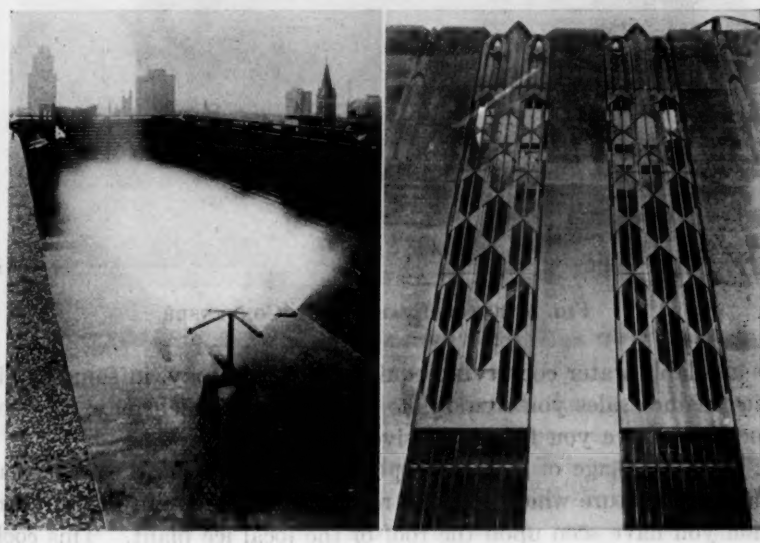


FIG. 4. THE OLD AND THE NEW IN COOLING TOWERS

demand. But, in certain cities, the cost of water was high and the manufacturers of air conditioning and refrigerating equipment sought means of reducing the water costs.

And, so, economic forces began to stimulate the development of equipment which would reduce sales resistance by reducing the need for water where your rates were high, or where the threat of water shortage might restrict its use. This has borne fruit, so that there

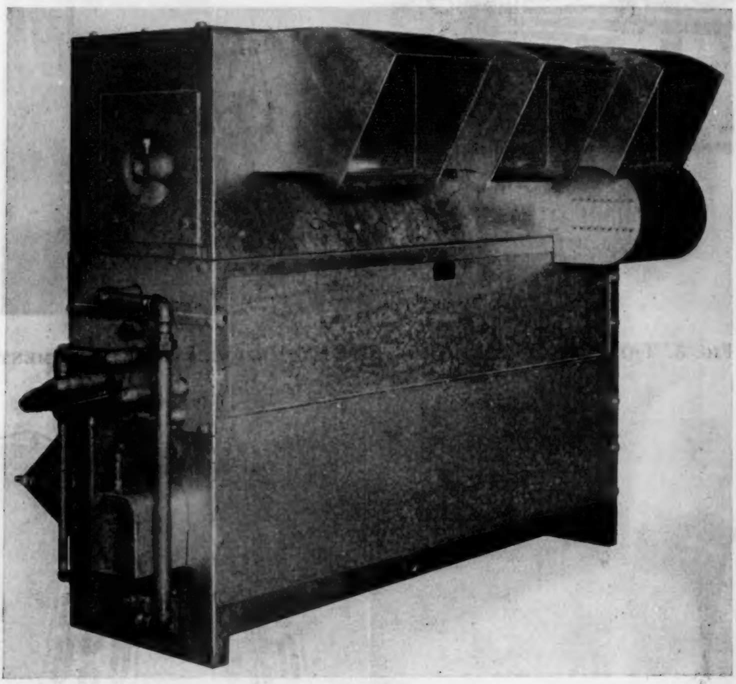


FIG. 5. 40-TON EVAPORATIVE CONDENSER

is available water conserving equipment which may, in some localities, reduce sales you would like to make but which may, in other localities, save you from excessive demands.

The first stage of this development was the cooling tower—that slatted structure which you will recognize as the architectural abortion you have seen upon the roof of the local ice plant. This cooling tower is a means of heat disposal. The water passing through it is recirculated, being pumped through the condenser where it re-

ceives heat, and then through the cooling tower where it gives off this heat to the air.

Some of the water is carried away by the wind, some of it is lost by evaporation, and that which is lost must be replenished from your mains. This loss is quite small, so that the use of this cooling tower for heat disposal cuts the demand for water to less than 10 percent of what it would otherwise be.

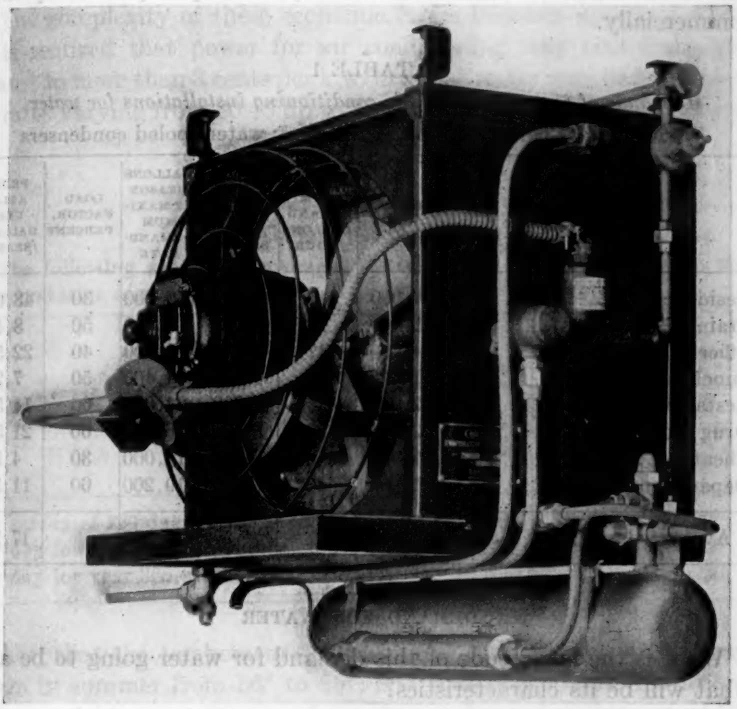


FIG. 6. 3-TON EVAPORATIVE CONDENSER

To be fair to the cooling tower, it must be said that much has been done to improve its appearance.

The second stage was the development of the evaporative condenser in which the functions of the condenser and the cooling tower have been combined. The features of water conservation have been slightly improved and the space occupied by equipment greatly reduced. This piece of equipment may be placed upon the roof of

a building and appear as a small rectangle, or, if it is feasible to lead air to and from it through ducts, it may be tucked away in one corner of the machinery room.

Since there is no windage loss from the evaporative condenser, it requires less than 5 percent of the water which you would otherwise furnish.

Both the tower and the evaporative condenser are instruments for the conservation of water and are fully developed practically and commercially.

TABLE 1

Demands of several types of air conditioning installations for water

Values are per person; season of 120 days; water cooled condensers

	B.T.U./HOUR	MAXIMUM DEMAND, GALLONS/HOUR	HOURS OPERATION PER SEASON	GALLONS/SEASON AT MAXIMUM DEMAND RATE	LOAD FACTOR, PERCENT	PROBABLE USE, GALLONS/SEASON
Residence	4,000	40	2,900	160,000	30	48,000
Train	1,400	14	1,200	16,800	50	8,400
Office	4,700	47	1,200	56,400	40	22,500
Lunch room	1,600	16	900	14,400	50	7,200
Restaurant	2,000	20	1,200	24,000	60	14,500
Drug store	2,000	20	1,800	36,000	60	21,500
Theatre	1,000	10	1,500	15,000	30	4,500
Department store	1,600	16	1,200	19,200	60	11,500
Average	2,300			42,500		17,200

DEMAND FOR WATER

What is the magnitude of this demand for water going to be and what will be its characteristics?

Let us examine the demands for water of the several types of prevalent installations of air conditioning, which are given in table 1.

B.T.U., or thermal units, per person will vary somewhat for each installation, but limits of variation are well known. Hours of operation per season can readily be set for maintenance of the desired conditions. Load factors, or the relation of average load to maximum load, are based on experience. Gallons of both maximum and average demand must be based on an average water temperature and an average balance between water and power.

Hence, table 1 is necessarily composed of figures which are admit-

tedly controversial as to the interpretation of experience, for multifarious economic forces press in some localities for excessive use of water and in others for conservation thereof.

For a given power rate, cold water may throw the balance in favor of heat disposal entirely by water—warm water may do the opposite. Low power rates may favor conservation of water—high power rates, the opposite. A combination of high's of water temperature and power cost operate strongly in favor of heat disposal by evaporation.

The complexity of these economic forces becomes apparent when it is realized that power for air conditioning may cost from less than 1 to more than 3 cents per k.w. hr.; that water may be purchased at rates varying from $0.45 up to $2.85 per 1000 cubic feet—perhaps

TABLE 2

Air conditioning requirements for water

Gallons per person

The following are "cold averages," admittedly controversial as to the interpretation of experience.

	NO EVAPORATIVE CONDENSERS OR COOLING TOWERS USED		BASED ON USE OF EVAPORATIVE CONDENSERS OR COOLING TOWERS ON 60 PERCENT OF INSTALLATIONS 1935 EXPERIENCE	
	If maximum demand is sustained	Probable use	If maximum demand is sustained	Probable use
Per season of 120 days	42,500	17,200	17,000	7,000
Per day for season of 120 days	350	140	**140**	60
Per day for year 365, days	120	50	50	**20**

higher; that the temperature of the water coming from mains may range in summer from 56° to 85°; and that pocketbook limitations as to available capital may enforce the purchase of equipment which is above average in demand for water or power, or both.

But, let's continue our examination of the averages arrived at in table 1 as they are carried forward in table 2. Three important facts become obvious.

First, that the demand for water is seasonal. It is high in summer and, as previously stated, insignificant in winter. A 120-day cooling season is typical of New York. In the South, it may exceed 150 days.

Second, that economic forces and restrictive regulations, such as

are now applied in New York, did, in 1935, lead to extensive purchase of evaporative heat disposal systems. As a result, water consumption was cut 60 percent.

Third, that within the cooling season, the peak demand is far above average consumption.

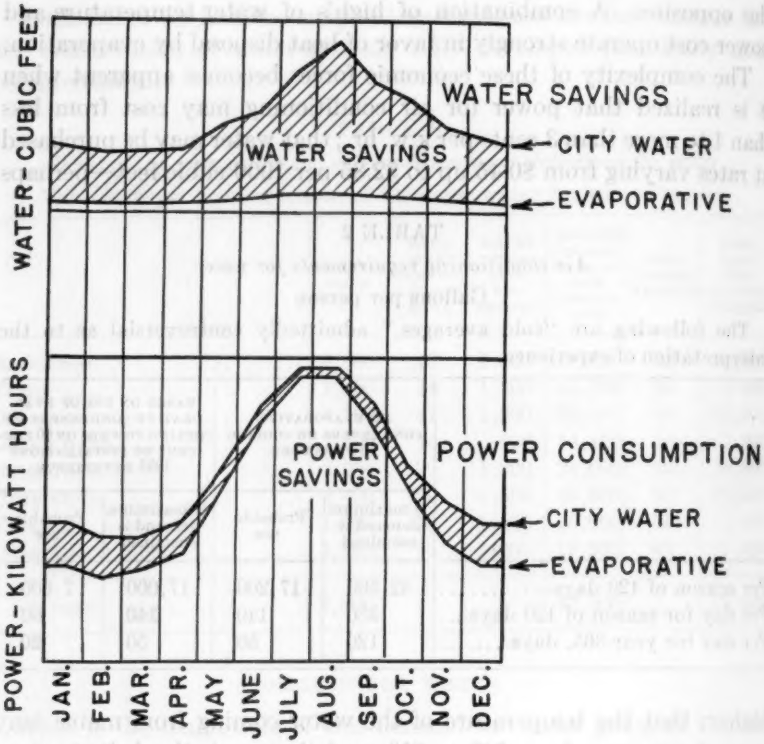


FIG. 7. WATER CONSERVATION MADE POSSIBLE BY USE OF EVAPORATIVE CONDENSERS

Two figures in table 2 are of such importance that they have been set in boldface type.

Note carefully that they are *not* the *per capita* consumption you are accustomed to using.

Air-conditioning-wise, they *will become per capita* consumption if and when air conditioning is installed and operated for the entire population. This does not mean that every building in a city must

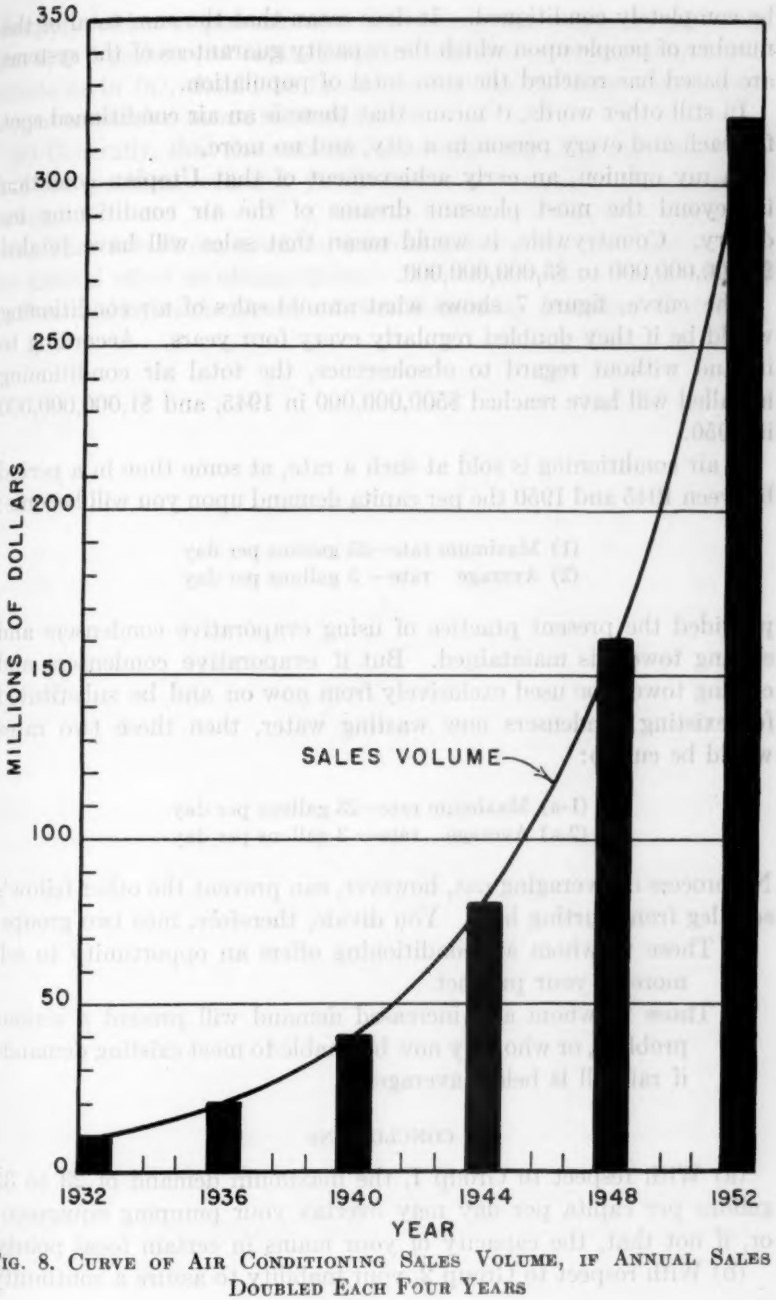


FIG. 8. CURVE OF AIR CONDITIONING SALES VOLUME, IF ANNUAL SALES DOUBLED EACH FOUR YEARS

be completely conditioned. It does mean that the sum total of the number of people upon which the capacity guarantees of the systems are based has reached the sum total of population.

In still other words, it means that there is an air conditioned spot for each and every person in a city, and no more.

In my opinion, an early achievement of that Utopian condition is beyond the most pleasant dreams of the air conditioning industry. Countrywide, it would mean that sales will have totaled \$4,000,000,000 to \$5,000,000,000.

The curve, figure 7 shows what annual sales of air conditioning would be if they doubled regularly every four years. According to it, and without regard to obsolescence, the total air conditioning installed will have reached \$500,000,000 in 1945, and \$1,000,000,000 in 1950.

If air conditioning is sold at such a rate, at some time in a period between 1945 and 1950 the per capita demand upon you will become:

(1) Maximum rate—35 gallons per day
(2) Average rate— 5 gallons per day

provided the present practice of using evaporative condensers and cooling towers is maintained. But if evaporative condensers and cooling towers be used exclusively from now on and be substituted for existing condensers now wasting water, then these two rates would be cut to:

(1-a) Maximum rate—23 gallons per day
(2-a) Average rate— 3 gallons per day

No process of averaging out, however, can prevent the other fellow's sore leg from hurting him. You divide, therefore, into two groups:

1. Those to whom air conditioning offers an opportunity to sell more of your product.
2. Those to whom any increased demand will present a serious problem, or who may now be unable to meet existing demands if rainfall is below average.

CONCLUSIONS

(a) With respect to Group I, the maximum demand of 23 to 35 gallons per capita per day may overtax your pumping equipment or, if not that, the capacity of your mains in certain focal points.

(b) With respect to Group 2, your inability to assure a continuity

of supply, either at present rates of consumption, or at present rates plus 3 to 5 gallons per capita per year average, or to pump and distribute as in (a), should be faced squarely as the facts stand, and proper restrictive measures instituted.

(c) Generally, that the number and complexity of factors are such that the problem will not yield, except to analysis of a specific survey.

(d) That it is to our common interest to know the facts and worth our mutual effort to obtain them.

(Presented before the New York Section meeting, December 27, 1935.)

WATER WORKS CONSTRUCTION IN NEW YORK STATE WITH FEDERAL AID

By Earl Devendorf
(*Associate Director, Division of Sanitation, State Health Department Albany, N. Y.*)

Since 1931 members of the American Water Works Association have played a prominent and important part in the promotion, planning, and supervision of water works construction as a means of relief of the large army of unemployed.

Although much has been written on the subject and all authorities have been in general agreement as to the merits of water works construction from the standpoint of permanent value to the country, so far as the writer is aware no one has presented a record of the accomplishments in this particularly field in New York State. It would seem fitting, therefore, and it will be the writer's endeavor to present a review of the record of accomplishments in the field of water works construction in particular under the various federal and state agencies created to assist local authorities in initiating work relief projects with federal and state aid.

With the creation of PWA, at the request of the State Advisory Board, the speaker was delegated by Dr. Thomas Parran, Jr., State Commissioner of Health, to coöperate with and assist Mr. Arthur S. Tuttle, then State PWA Engineer, and now Acting State Director for PWA, in passing on applications for Federal Aid covering sanitary projects submitted by municipal authorities in New York State. Subsequently the speaker was appointed by Governor Lehman to assist municipal authorities having projects pending before the Washington office of the PWA.

Similarly with the creation of the State TERA, at the request of the Administration, the speaker was likewise delegated to assist the Administration in the undertaking of essential needed sanitary projects for the relief of the unemployed.

More recently the speaker has been appointed State Consultant on Sanitation by Mr. Lester Herzog, State WPA Administrator, and

is now actively engaged in assisting in passing on sanitary projects pending before the WPA for approval.

Without the wholehearted assistance of Mr. C. A. Holmquist, Director of the Division of Sanitation of the State Health Department and his entire staff it would have been impossible for the speaker to carry out the large amount of detailed work necessary with any degree of effectiveness. Similarly the coöperation and assistance received both from Mr. Tuttle of the PWA and his entire staff, and from Mr. James P. Lee, Director of the Project Division of the TERA and his engineering staff, have been large factors in the successful and satisfactory manner in which water works and other sanitary projects have been completed in New York State during the past four years, as indicated from the data presented here below. Time, of course, will not permit going into details regarding individual projects.

TABLE 1

TYPES OF PROJECTS	NUMBER	COST	PERCENT OF TOTAL	CONTRACTS AWARDED UP TO OCTOBER 25, 1935	PERCENT AWARDED
Old PWA program					
Water Works	35	$5,780,000	4.7	4,160,000	72
Sewage and sewage disposal	33	10,700,000	8.7	8,550,000	80
Total, all types of projects, $123,000,000					
New PWA program					
		(AS OF DECEMBER 10, 1935)			
Water Works	18	$1,380,000	1.12		
Sewage and sewage disposal	25	48,000,000	39.0		
Total, all types of projects, $123,500,000					

PWA

The accomplishments in water works sanitation projects under the PWA in New York State are summarized in table 1. These data were obtained from the office of Mr. Arthur S. Tuttle, Acting State Director of PWA.

From this table it is readily seen that water works construction

comprised but a small proportion of the total amount of construction projects undertaken under PWA. This is partly due to the time required by municipal authorities to authorize and have surveys and plans prepared for such construction projects. Included among these waterworks projects are many improvements and enlargements that have long been recommended by the State Department of Health.

TERA

From Mr. James P. Lee, Director of the Project Division of TERA the following information regarding water works construction was obtained which is most interesting and certainly enlightening to all of us to indicate the vast amount of both money and wages which

TABLE 2

Cost of projects undertaken by TERA in New York State and man-days of work performed November 1, 1931 to November 20, 1933

CLASSIFICATION	NUMBER OF PROJECTS	TOTAL COST	PER CENT TOTAL COST	RELIEF WAGES	OTHER COSTS	PER CENT		MAN-DAYS OF WORK PROVIDED
						Wages	Other	
All projects	4,866	113,901,948.29	100	95,649,911.72	18,252,036.57	84	16	23,467,580
*Sanitation	697	11,049,675.59	9.7	8,529,750.40	2,519,925.19	77	23	2,141,560
Water supply	331	3,929,288.53	3.5	2,693,613.78	1,235,674.75	69	31	605,730

This table covers work done prior to the CWA.

* Sanitary and storm sewers, sewage disposal plants, incinerators, etc.

have gone into the water works construction during the past three years. It is certainly to the credit of Mr. Lee and the Administration staff of the TERA that they have been able to present such detailed and complete breakdowns of the cost of the various projects undertaken as indicated in tables 2 to 6, inclusive.

At this point it should be recorded that shortly after beginning operation the TERA organized an engineering personnel with district engineers appointed throughout various sections of New York State to promote and supervise the carrying out of construction projects. The effect of such engineering supervision was noteworthy and resulted not only in an improvement in the types of projects undertaken but also in improved efficiency in the construction methods employed.

Table 2 shows the summation of work projects undertaken by the TERA in New York State beginning with the formation of the

TABLE 3

*Expenditures under CWA November 20, 1933 to March 31, 1934**

CLASS OF WORK	NUMBER OF PROJECTS	MAN-HOURS OF WORK PERFORMED	WAGES AND SALARIES	TEAMS, TRUCKS AND EQUIPMENT	MATERIALS	OTHER COSTS	TOTAL COSTS
New construction of sewers, drainage and sanitation, sewage disposal plants, incinerators	321	9,935,249	6,055,832.73	310,612.53	2,110,803.83	84,342.32	8,561.591.41
Repair and maintenance of sewers, drainage, etc.	99	1,392,473	829,395.96	36,023.15	176,580.32	4,138.77	1,046,138.20
New construction waterworks and other public utilities, incinerators, etc.	147	1,693,226	968,298.73	62,908.02	794,654.14	24,027.32	1,849,888.21
Repair and maintenance of waterworks, etc.	96	1,976,660	1,189,346.76	48,666.34	262,890,22	4,948.87	1,505,852.19
Eradication and control of disease bearers (Dutch Elm disease)	3	79,770	66,466.07	71.62	335.02	71.96	66,944.67
Eradication and control of pests (mosquitoes)	20	2,918,158	1,694,542.01	3,117.37	24,329.72	2,152.43	1,724,141.53
Eradication and control of poisonous plants (poison ivy, oak, etc.)	0						

* Total expenditures under CWA for all classifications of work performed are not available at this time.

TABLE 4

*Amounts approved on projects by the TERA from April 1, 1934 to July 1, 1935**

Class of work or service	Number of projects or jobs	Man-hours of work performed	Wages and salaries	Teams, trucks and equipment	Materials	Others costs	Total costs	Percent total costs
Total all projects	16,188	545,999,424	322,543,547	27,572,940	61,981,578	5,939,553	418,037,618	
New construction of sewers, drainage and sanitation, sewage disposal plants, incinerators	934	29,127,919	16,119,534	2,017,476	4,982,917	480,982	23,600,909	5.65
Repair and maintenance of sewers, drainage, etc.	158	6,003,759	1,972,724	187,970	154,977	16,018	2,331,689	0.56
New construction waterworks and other public utilities, incinerators, etc.	538	10,521,690	5,962,172	512,648	3,457,217	312,392	10,244,429	2.46
Repair and maintenance of waterworks, etc.	235	8,067,611	4,436,085	361,200	935,589	24,457	5,757,331	1.38
Eradication and control of disease bearers (Dutch Elm disease)	4	148,213	123,628	15,853	14,415	1,578	155,474	0.04
Eradication and control of pests (mosquitoes)	90	8,588,641	3,938,916	159,993	114,265	42,300	4,255,474	1.02
Eradication and control of poisonous plants (poison ivy, oak, etc.)	1	33,072	17,141		277		17,418	0.004

* No classification of expenditures is yet available.

TABLE 5

An average weekly payroll subsequent to the CWA program

	RELIEF		NON-RELIEF		TOTAL	
	Man-hours	Wages	Man-hours	Wages	Man-hours	Wages
Total all projects	3,724,275	$2,325,323	1,008,371	$681,501	4,732,647	$3,006,824
New construction of sewers, drainage and sanitation, sewage disposal plants, incinerators	277,710	119,985	2,455	2,084	230,165	122,069
Repair and maintenance of sewers, drainage, etc.	35,078	19,255	110	104	35,188	19,359
New construction waterworks and other public utilities, incinerators, etc.	94,335	49,347	1,431	1,100	95,766	50,447
Repair and maintenance of waterworks, etc.	99,468	53,009	851	629	100,319	53,638
Eradication and control of disease bearers (Dutch Elm disease)	Negligible					
Eradication and control of pests (mosquitoes)	29,094	14,722	411	371	29,505	15,093
Eradication and control of poisonous plants (poison ivy, oak, etc.)	9,816	5,030	24	17	9,840	5,047

TERA in November, 1931 and extending for two years to November, 1933 and covers the work done prior to CWA. Of a total of about 114 million dollars spent only 4 million dollars went for water works construction, equivalent to about 3.5 percent of the total of all types of construction.

A classification of expenditures undertaken under CWA and covering the period from November 20, 1933 to March 31, 1934 is given in table 3. The total expenditures under this program for all

TABLE 6

Work accomplished under TERA to July 1, 1935*

Miles of sanitary sewers constructed	273
Miles of storm sewers constructed	233
Miles of water mains constructed	273
Miles of mosquito-control drainage ditches constructed	1,942
Miles of water lines (small pipe) installed	26
Miles of sewers cleaned	27
Number of reservoirs involving construction	61
Number of fire protection reservoirs constructed	47
Number of reservoirs improved and repaired	33
Number of cesspools constructed	77
Number of sanitary privies constructed	15
Number of sewage disposal plants constructed (including septic tanks)	23
Number of sewage disposal plants improved and repaired	25
Number of garbage disposal plants constructed	23
Number of garbage disposal improved and repaired	4
Number of wells dug	23
Number of wells improved	4
Number of fire hydrants installed	249
Number of fire hydrants repaired	293

* Some of this work was started under CWA but most of it has been accomplished after April 1, 1934.

classification of work are not available at this time. 147 new water works construction projects were carried out at a total cost of about $1,850,000. 96 repair and maintenance water works projects were carried out at a cost of about a million and a half dollars during this period.

A tabulation of work relief carried out under the TERA from the close of CWA program from April 1, 1934 to July 1, 1935 is given in table 4. Over 16,000 work relief projects were undertaken at a total cost of $418,000,000. Of this amount 538 new water works

construction projects were completed at a total cost of slightly over $10,000,000 amounting to a little less than 2.5 percent of the total. Two hundred and thirty-five repair and maintenance water works projects were completed at a cost of a little over $5\frac{3}{4}$ million dollars amounting to a little under 1.5 percent of the total.

Table 5 represents an average weekly payroll carried out in the period covered under table 4; namely from the close of the CWA to the first of July, 1935.

Table 6 gives a summary of sanitary work carried out under TERA during the period covered under table 4; namely from the close of CWA to July 1, 1935. About 500 miles of sanitary and storm sewers were constructed and 275 miles of water mains.

Obviously it would be impossible within the short time available to discuss any details of individual projects. A few outstanding projects have been selected as representative of various types of projects which have been undertaken and completed under the TERA. The information regarding these projects has been furnished by Mr. Lee, Director of the Project Division of the TERA.

WATER SUPPLIES

Many water supply main construction projects have been completed under the TERA. One of the most noteworthy and favorable in this field of public improvement is found in the city of Buffalo's project for the installation of high-pressure water mains. This project was started in January, 1934 and called for a total of 10 miles of pipe line. This project was carried out in accordance with plans of Mr. Alan Drake, Commissioner of Water, and complied with the demand of the board of fire underwriters for improved water supply distribution. At the time of reporting some seven miles of 12, 15- and 20-inch universal high-pressure pipe were laid and the daily average number of men employed during the period has been 450. The approximate cost was reported as $810,000 of which 75 percent was paid for over 900,000 man-hours of work. The work was accomplished in the face of many unforseen obstacles due to the existence of other lines laid by the utilities companies. It was reported that the total cost ran 12 percent under the original estimate.

RESERVOIR IMPROVEMENTS

Many water supply reservoirs have been undertaken by TERA. One of the outstanding examples is that of the city of Plattsburg.

TABLE 7

Water supply

PLACE	DESCRIPTION	COST
1933 PWA program		
Goshen	Filter plant and additions reservoir	$116,900
Groton	Improvement to water supply and purchase of system	138,000
Pelham	Water supply system	152,800
Holland	Water supply system	79,000
Mt. Morris	Modification of filtration plant	45,000
Watervliet	New supply main	215,000
Gloversville	New reservoir and main	236,000
Saranac Lake	Water mains	9,600
Almond	New water supply system	33,200
Plandome	Water supply system	68,700
White Plains	Additional water mains	45,300
Buchanan	Water supply system	103,000
Plattsburg	Water mains	25,580
Pleasantville	Additional water mains	8,400
Irvington	Alterations and additions	42,000
Sharon Springs	Filter plant improvements	33,900
Stillwater	Water supply system	100,000
Wells	Water supply system	65,000
East Rochester	Well and tank storage	80,000
Elba	Waterworks system	37,000
Hamburg	Additional well, aerator and reservoir	50,000
Avon	Water main extension	6,692
Bethlehem Water Dist.	New reservoir and water softening plant	80,000
Niagara Falls	Filter plant improvement	445,000
New York City	Water tunnel No. 2	1,000,000
Yonkers	Water main extensions	297,000
New York City	Water works improvements	1,013,000
Plattsburg	Water filtration plant	50,000
Youngstown	Filtration plant	41,000
New York City	Water works improvements	540,000
1935 PWA program		
Webster	Addition to water system	212,000
Ithaca	Extensions	35,000
Carmel	Water system	50,909
Scio	Water system	49,000
Carmel	Water supply system	110,000
Perry	Water tank	40,650
Mt. Kisco	Water system	43,000

TABLE 7—*Concluded*

PLACE	DESCRIPTION	COST
	1935 PWA program—*Concluded*	
Mt. Kisco	Wells	62,000
Red Hook	Water works improvements	63,892
Rochester	Dam	320,000
Rochester	Spillway	130,000
Ransomville	Water works	62,150
Lockport	Water mains	60,000
St. Regis Falls	Water supply system	115,306
Oswego	Water mains	64,350
Colonie	Water extensions	62,000
Rockville Center	Improvement to water supply system	182,000
Oneida	Filter plant	76,140
Center Island	Water supply	25,200
Cortlandt, Roe Park Water Dist.	Water supply system	32,000

where concrete bottoms and linings and protective walls were constructed in the existing distributing reservoir. The capacity of this reservoir has been increased by 1.6 million gallons providing a total storage capacity of 4.5 million gallons.

This improvement provides the city of Plattsburg with additional protection of its water supplies and has been recommended by the State Department of Health for many years to say nothing of the increased fire storage made available.

WATERSHED IMPROVEMENTS

Many watershed improvements have been carried out including reforestation and elimination of fire hazards and insect pests. Much of the improvements to existing reforestation areas has been carried out under forestry experts of the State Conservation Commission.

As an example of such improvement the city of Middletown may be cited where the eradication of insects has been accomplished. The insects had threatened to ruin the whole of over 600,000 trees on the Middletown watershed area. In addition some 80,000 small pine trees were planted and the repairing and building of trenches was accomplished. This type of project required a small amount of equipment and materials, 95 percent of the $33,500 outlay for this improvement going for wages.

WPA

Since the WPA only began operation December 1, 1935 it is entirely too early to present any summary of its accomplishments on water works construction. Those projects in New York State which have received Presidential approval include some 10 million dollars of water works construction. Of this amount projects involving new construction totaling slightly over 2 million dollars in estimated cost were released and under operation as of December 15, 1935.

SUMMARY

These tabulations indicate the vast amount of public funds that have been expended in New York State during the past four years of the depression as a means of relief of unemployment. Under PWA some $245,000,000 in New York State have been alloted for public works construction. Under TERA and CWA programs nearly three-quarters of a billion dollars have been expended. One cannot predict how much longer it will be necessary to carry on public construction projects as a means of relief of unemployment. Although there is no question but business conditions in general are greatly improved and have resulted in placing many men in permanent positions of employment, there is still a vast amount of unemployment in the country and there seems to be a general agreement that industry will be unable to absorb the unemployed at least in the immediate future. It seems probable, therefore, that work relief will continue in some form during the coming year. All engineers and water works officials should have plans prepared for needed water works construction in order to be able to initiate the work without delay.

The small percentage of relief funds which have been expanded for water works construction is, in my opinion, an indictment of engineers and water works officials. It is certain there is no public works construction which is of more permanent and lasting value and benefit to any community than water works construction if well planned.

Under the National Resources Committee (formerly National Resources Board) State Planning Boards were organized in 46 states for the purpose of coördinating and planning needed water works projects from the national, state and local viewpoints. Leading engineers of the country are represented on these boards and it seems not only logical but essential that the recommendations of these organizations for water works improvements should be given

careful consideration in planning future work programs with federal or state aid.

In conclusion, I again urge that we all study our own local needs for water works construction, have plans prepared for the undertaking of such needed improvements and continue to urge and follow up with your city administration leaders and impress on them the importance of including water work construction as a major part of any future work program.

(Presented before the New York Section meeting, December 27, 1935.)

STATISTICAL COMPARISON OF WATER DEPARTMENTS OF WISCONSIN CITIES

By E. W. Moke

(Public Service Commission, Madison, Wis.)

By its very nature a statistical paper must be limited in scope in order that one can grasp and retain some of the information presented. The particular figures your program committee requested were data on operating and turn-over ratios, depreciation, taxes and return, and the percentage distribution of operating revenue collected from the major classes of water service.

GENERAL RATIOS OF WATER WORKS OPERATION

A word of explanation should be given at first outlining the scope of these figures. The data are taken from the annual reports to the Commission for the years 1933 and 1934. All communities having over 3,000 population were included. This group includes 72 cities and villages which are served by 65 municipally owned plants and 7 privately owned plants. However, in order to make the figures comparable it was found necessary to eliminate 5 utilities which purchased all or part of their water supply at wholesale. These 5 were excluded from the general ratios of water works operation but not from the figures on distribution of operating revenues. On the other hand, about 25 other utilities were excluded from the group for which we show the distribution of operating revenues because this classification of revenues was not shown or required in the annual reports.

Operating ratio

First, let us take the operating ratio. (For summary see table 1.) For the purposes of this study this has been defined as the percentage of operating revenues used for operating expenses *exclusive* of depreciation and taxes, and, in the case of privately owned plants, of uncollectible revenues. The simple average of operating ratios, as above defined, for 1933 and 1934, was 35.4 percent for plants serving cities ranging in size from 20,000 to 68,000; 39.2 percent for plants

serving cities from 8,000 to 20,000; 42.3 percent for plants serving cities from 4,000 to 8,000; and 43.1 percent for plants serving cities from 3,000 to 4,000.

A simple average is affected by a few extreme cases. What is known as a median average, namely the figure which is the middle item between the highest and lowest, minimizes the effect of the extremely high and extremely low figures. The average operating ratios for each group of cities, using this median, would be in most cases from 1 to 2 points less than the simple average which is given.

TABLE 1

Operating ratio—municipally owned water plants

	POPULATION GROUP			
	20,000–68,000	8,000–20,000	4,000–8,000	3,000–4,000
	percent	*percent*	*percent*	*percent*
1933				
Highest ratio	44.3	53.3	113.7	90.1
Lowest ratio	25.9	26.1	15.8	22.5
Range	18.4	27.2	97.9	67.6
Average	34.3	38.4	42.1	43.3
1934				
Highest ratio	50.9	62.6	110.8	93.7
Lowest ratio	26.0	25.3	22.0	17.1
Range	24.9	37.3	88.8	76.6
Average	36.4	39.7	42.5	42.9
1933 and 1934				
Simple average	35.4	39.2	42.3	43.1

It is interesting to note the range of these operating ratios. For example, in the first population group, from 20,000 to 68,000, the highest operating ratio was 44.3 percent in 1933 and 50.9 percent in 1934, whereas, the lowest ratio was 25.9 percent in 1933 and 26 percent in 1934. In other words, there was a range of approximately 18 points between the highest and lowest operating ratio in 1933 and 25 points in 1934 for this size of plant. I will not weary you with the detailed figures for the other sizes of plants. Our figures show, however, that as the size of plant decreases the higher becomes the highest operating ratio and the greater becomes the range between

high and low. Generally speaking, the range between the highest and lowest operating ratio in every size of plant, except one, was higher in 1934 than in 1933. It is interesting also to note that the average operating ratio in 1934 was slightly higher than in 1933 with the exception of utilities serving communities of from 3,000 to 4,000. This would seem to indicate a tendency for operating expenses to increase somewhat faster than operating revenues, comparing only 1934 and 1933.

The difference between water plants of different sizes are worthy of note. In general, both in 1933 and in 1934, the operating ratio,

TABLE 2

Turnover ratio—municipally owned water plants

	POPULATION GROUP			
	20,000–68,000	8,000–20,000	4,000–8,000	3,000–4,000
	percent	*percent*	*percent*	*percent*
1933				
Highest ratio	11.23	12.25	14.69	16.47
Lowest ratio	6.84	6.68	6.62	5.22
Range	4.39	5.57	8.07	11.25
Average	8.81	8.79	9.44	9.86
1934				
Highest ratio	11.72	11.79	12.98	16.24
Lowest ratio	6.85	6.79	6.80	4.69
Range	4.87	5.00	6.18	11.55
Average	8.82	8.73	9.26	9.55
1933 and 1934				
Simple average	8.82	8.76	9.35	9.70

as above defined, increases as the size of the plant decreases. This is indicated from the average figures previously given.

The sample group of utilities includes 7 privately owned utilities. we have compared the average operating ratios for municipal and for private water plants in the two years here taken. For municipal plants the average operating ratio of all sizes of plants was 39.7 percent in 1933 and 40.5 percent in 1934, whereas, the corresponding figures for the privately owned plants were 38.8 percent in 1933 and 40.9 percent in 1934. Expressed another way, although there is

very little difference between the operating ratios themselves, the municipally owned plants in 1934 showed a substantially smaller increase in operating ratios than was the case for the privately owned plants.

Turnover ratio

The second item I was asked to compile figures on is the turnover ratio. (For summary see table 2.) We have defined that to mean the average investment in property and plant for each dollar of annual revenue. In other words, it represents the number of years required for aggregate revenues to equal the investment in property and plant. For this purpose we have used the simple average of the opening and closing gross property and plant figures without deduction of depreciation reserves, contributions, etc.

The ratio is a very significant one in an industry requiring large amounts of fixed investment. Where the turnover ratio is high, fixed charges, such as depreciation, taxes, and interest, become an almost controlling element in the total cost of service. To meet these heavy fixed charges it therefore requires a relatively low operating ratio if the enterprise is successful. The higher the turnover ratio the lower the operating ratio should be for a successful enterprise.

For the two years here included the simple average turnover ratio was 8.82 for utilities in cities of from 20,000 to 68,000; 8.76 in cities of from 8,000 to 20,000; 9.35 in cities of from 4,000 to 8,000; and 9.70 in cities of from 3,000 to 4,000. In other words, in these two years it requires $9.00 to $10.00 of fixed capital investment to produce a dollar of revenue. I should explain at this point that one municipal utility reported plant figures on the basis of an appraisal rather than historical cost and the difference between these two figures has been eliminated. Also, all the privately owned water utilities have been excluded, because in most cases a property and plant figure for each community served was not reported.

The highest turnover ratio found in the two years was 16.4 in 1933 in the case of a plant serving a community between 3,000 and 4,000. The lowest turnover ratio was 4.69 in 1934, also a plant serving a community from 3,000 to 4,000. In general, the range between the lowest and the highest turnover ratios increased in both years as the size of the plant *decreased.* In other words, the smaller the plant the greater the range between high and low turn-

over ratios. Comparing both years together the range between high and low ratios in the case of the smallest plants was more than twice the range in the case of the largest plants.

In general, water plants in communities of 20,000 or less made a better showing in 1934 than in 1933 in improving their turnover

TABLE 3

Depreciation—percent of property and plant—municipally owned plants

	POPULATION GROUP			
	20,000–68,000	8,000–20,000	4,000–8,000	3,000–4,000
	percent	*percent*	*percent*	*percent*
1933				
Highest rate	2.01	2.00	2.36	2.00
Lowest rate	0.59	1.22	0.34	0.98
Range	1.42	0.78	2.02	1.02
Average	1.37	1.49	1.39	1.53
1934				
Highest rate	2.03	2.00	2.29	2.05
Lowest rate	0.59	1.22	0.34	1.01
Range	1.44	0.78	1.95	1.04
Average	1.38	1.47	1.40	1.51
1933 and 1934				
Simple average	1.38	1.48	1.40	1.52

Percent of gross revenue, 1934

	POPULATION GROUP			
	20,000–68,000	8,000–20,000	4,000–8,000	3,000–4,000
	percent	*percent*	*percent*	*percent*
Municipal	12.4	13.1	12.6	14.8
Private	9.0	4.1	4.0	10.6

ratios. The plants serving communities of from 20,000 to 68,000 show almost the identical turnover ratio in 1934 as in 1933.

Depreciation

Depreciation is a subject of very vital interest to all as well as in industries requiring large amounts of fixed capital. For these

figures we have used the ratio of reported annual depreciation expense to the average property and plant. (For summary see table 3.) The average property and plant is the gross plant value as reported for the beginning and the end of each year. It should be recognized that in the case of a water plant much of the property is long-lived. It is also well known that the depreciation expense required is materially affected by the standards of maintenance, as well as, of course, the accounting for depreciation as distinct from maintenance. In general, the higher the standards and level of maintenance expense, and the longer lived the property, the lower will adequate depreciation expense be.

For the years 1933 and 1934 together the depreciation expense reported was as follows:

Population classes	*Depreciation expense ratio, percent*
20,000 to 68,000	1.38
8,000 to 20,000	1.48
4,000 to 8,000	1.40
3,000 to 4,000	1.52

The general tendency of the depreciation ratio is upward as the size of the plant decreases with the exception of the group of plants serving communities of from 4,000 to 8,000. However, it is also noticeable that there is very little difference in the percentages. The range from the lowest to the highest average is only 0.14.

The highest depreciation ratio found was 2.36 percent in a plant serving a community between 4,000 and 8,000. This was in 1933. The highest in 1934 was 2.29 percent for the same plant. The lowest depreciation ratio found was 0.34 of 1 percent, also in the case of a plant serving a community between 4,000 and 8,000. The range between high and low depreciation expense in the plants of various sizes shows no consistent relationship. In general, plants serving communities from 8,000 to 20,000 population appear to be by far the most consistent in their depreciation rates since the range from low to high is smallest in this group.

There is very little change in the average depreciation rates comparing 1933 with 1934.

In a great majority of cases these depreciation rates are what we call composite or flat depreciation rates. In other words, only a very few water utilities determine depreciation expense by applying to each major class of property a separate rate of depreciation. In

fact only two of the Class A and B municipal utilities show in their reports that they determine depreciation expense by the application of class depreciation rates. Of course, the most accurate method of determining depreciation expense would be by the application of class rates. This is because the proportion of the total fixed capital in short-lived and long-lived plant will vary somewhat from time to time. Also, it should be borne in mind that a depreciation rate adequate for one water utility may not be adequate for another water utility because of differences in the proportions of long-lived and short-lived plant. For example, one water plant I have in mind has a large proportion of its total fixed capital in structures and source of water supply facilities which on account of their present operation is likely to prove a short-lived investment because of either inadequacy or obsolescence. Such a plant would therefore tend to require a somewhat higher composite depreciation rate than another plant not having these characteristics.

Many of you are intensely interested in the determination of proper depreciation rates. It should be pointed out, however, that for the foregoing reasons a depreciation rate should be determined for each individual property in accordance with local circumstances and conditions. Assuming they may be of some interest to you the recommendations by the Engineering Department of the Commission of the usual range of service lines and depreciation rates for major classes of property are summarized in table 4. These data were prepared sometime ago. Let me reiterate that they do not necessarily apply in every case, but they may serve as a general guide for your consideration.

The above figures on depreciation do not include privately owned water plants. As previously explained, most of the privately owned water utilities do not report for each community served the classification of fixed capital. An alternative way of expressing depreciation expense is as a percentage of gross revenue. We have therefore compiled the ratios of depreciation expense to gross revenue for the municipally owned plants and the privately owned plants in the various population groups. These figures are included in table 3. In general, table 3 shows that the municipally owned plants provide for depreciation a larger proportion of gross revenue than do the privately owned plants. In some cases the difference is very material; in other cases the difference is not so substantial. On the whole, the very smallest and the very largest privately owned plants

TABLE 4

*Tables of approximate lives to be assigned to various types of structures and equipment**

	LIFE IN YEARS	AVERAGE
Filters		
Slow sand or gravity		100
Mechanical pressure	30–50	
Mechanical gravity		40
Wooden	20–30	25
Concrete		100
Hydrants	50–100	75
Mains		
Cast iron mains—including specials		100 up
Large riveted and lock bar steel mains	30–50	40
Small wrought iron or steel mains—black	20–40	30
Small wrought iron or steel mains—galvanized	30–50	40
Manholes—distribution system, same life as pipe on which located		
Meters	20–30	25
Meter boxes		
Wooden	10–20	15
Vitrified or iron	25–45	35
Reservoirs		
Earth embankment		100 up
Stone masonry	50–100	75
Reinforced concrete		100 up
Services		
Lead		100 up
Galvanized	30–50	40
Wrought iron or steel	20–40	30
Standpipes and steel tanks on steel towers	35–75	50
Tanks		
Wooden, on wooden towers	15–25	20
Wooden, on steel towers	30–50	40
Pressure tanks buried	25–50	40
Valves	50–100	75
Wells		
Driven or drilled	50–100	75
Large open, stone or brick walled	50–100	75
Generators and motors, (including motor generators, rotary converters and frequency changers)	20–30	25
Pumps		
Centrifugal and rotary	15–25	20
Geared power	20–30	25

* These lives and averages represent usual conditions found. It should be recognized that in particular situations these lives might not apply owing to unusual conditions.

TABLE 4—*Concluded*

	LIFE IN YEARS	AVERAGE
Pumps		
Steam-direct acting	20–30	25
Crank and flywheel	25–40	30
Shafting, pulley, etc.	20–25	22
Steam turbines	20–30	25
Power stations		
Frame	20–40	
Brick	35–50	
Fireproof	60–100	
Office buildings		
Frame	30–50	
Brick	50–75	
Fireproof	75–100	

provide for depreciation more adequate proportions of gross revenue than do the private plants of medium size.

Taxes

Taxes at the present time constitute an item of almost burning concern to municipal plant officials and city officials because of the pressure upon local budgets. Our figures on taxes are simply the ratio of annual tax accrual to the property and plant account at the beginning of the year. (For summary see table 5.)

In general for the two years 1933 and 1934, the tax expense averaged 1.58 percent of property and plant for communities of from 20,000 to 68,000; 1.72 percent in communities of from 8,000 to 20,000; 1.60 percent in communities of from 4,000 to 8,000; and 1.31 percent in communities of from 3,000 to 4,000. There is not much change between 1933 and 1934 but the general tendency is for 1934 tax accruals to be higher than for 1933. The increase was somewhat greater in the smallest than in the largest plants.

The highest tax expense accrual found was 3.51 percent and the lowest was 0.39 percent both being in the year 1934 in the population group of from 4,000 to 8,000. This statement excludes Milwaukee Water Works from consideration, as in all the above figures. In general, the range between the lowest and the highest tax accruals increased as the size of the plant decreased, with the exception of the smallest size of plant here considered.

In any discussion of public versus private ownership of utility property the item of taxes is a subject of controversy. As in the case of depreciation expense we have not compiled figures on the ratio of taxes to property and plant for privately owned water plants. Accordingly, the only comparison between private and municipally owned plants in the matter of taxes must be on the basis of a percentage of gross revenue. Such a comparison is shown in table 5. This table shows that on the average the privately owned utilities pay a larger proportion of each dollar of gross revenue in taxes than do the municipally owned utilities. On the average, the municipally owned utilities accrue from 13 to nearly 16 cents of each dollar of revenue for taxes, whereas the privately owned utilities accrued on

TABLE 5

Taxes—percent of property and plant—municipally owned plants

	POPULATION GROUP			
	20,000–68,000	8,000–20,000	4,000–8,000	3,000–4,000
	percent	*percent*	*percent*	*percent*
1933				
Highest rate	2.15	2.39	2.37	2.05
Lowest rate	1.06	1.00	0.40	0.40
Range	1.09	1.39	1.97	1.65
Average	1.59	1.67	1.53	1.21
1934				
Highest rate	2.10	2.71	3.51	2.74
Lowest rate	1.03	1.13	0.39	0.40
Range	1.07	1.58	3.12	2.34
Average	1.57	1.76	1.66	1.40
1933 and 1934				
Simple average	1.58	1.72	1.60	1.31

Percent of gross revenue, 1934

	POPULATION GROUP			
	20,000–68,000	8,000 20,000	4,000–8,000	3,000–4,000
	percent	*percent*	*percent*	*percent*
Municipal	13.7	15.8	14.7	12.9
Private	18.2	20.1	21.4	13.2

the average from 13 to 21.5 cents of each dollar of revenue for taxes. However, a municipally owned water plant has the distinction of showing the highest percentage of gross revenue accrued for taxes of any privately or publicly owned plant included in this study.

Return

For this study we have defined return on investment as the ratio of operating income to average property and plant less the reported depreciation reserve and customer contributions. Working capital has been excluded. Consequently, the returns shown tend to be somewhat higher than the returns on a rate base including working capital.

As might be expected, the rate of return for the water plants considered individually shows the widest fluctuations. On the average for the years 1933 and 1934, the rate of return was as follows for the size of municipal plants here considered.

Population classes	*Percent return*
20,000 to 68,000	5.99
8,000 to 20,000	4.89
4,000 to 8,000	4.45
3,000 to 4,000	4.73

The above figures exclude plants purchasing their water supply. The effect of this exclusion is to increase the average rates of return shown, although the difference is not very great. In general, the average percentage return did not change materially from 1933 to 1934. In one group of plants, namely the 3,000 to 4,000 population group, there was a small increase in the percent rate of return in 1934 compared with 1933: in the other groups there was a small decrease. The average percent rates for the water plants in communities of 20,000 population or less was about the same for each size group, whereas, the percent return for plants in communities above 20,000 was on the average about one point higher.

SUMMARY OF GENERAL RATIOS

By way of summarizing this portion of our report, we tabulate the two-year average of the general ratios previously discussed for each size of plant in table 6. On the whole, the operating and turnover ratios tend to increase as the size of plant decreases. On the other hand, the depreciation ratios do not appear to vary very much in plants of different sizes. Except for the largest plants, tax ac-

cruals apparently tend to decrease with the size of the plant. Except for the smallest size of plant, the percent return tends to decrease with the size of the plant.

DISTRIBUTION OF OPERATING REVENUES BY CLASSES OF SERVICE

Your program committee also asked us to compile figures showing what proportions of operating revenues were paid by the different

TABLE 6

Summary of statistical ratios—all municipals, excluding those that purchase water wholesale. Simple average for 1933 and 1934

POPULATION GROUP	STATISTICAL RATIOS				
	Operating ratio	Turn-over	Depreciation	Taxes	Return
	percent	*percent*	*percent*	*percent*	*percent*
20,000 to 68,000	35.4	8.82	1.38	1.58	5.99
8,000 to 20,000	39.2	8.76	1.48	1.72	4.89
4,000 to 8,000	42.3	9.35	1.40	1.60	4.45
3,000 to 4,000	43.1	9.72	1.52	1.31	4.73

TABLE 7

Percent distribution of revenue, 1934—Class A and B utilities reporting complete breakdown of revenue, excluding residential suburbs

	POPULATION GROUP		
	20,000–68,000	8,000–20,000	3,000–8,000
	percent	*percent*	*percent*
Residential service	42.2	44.8	44.1
Commercial service	13.3	12.0	15.1
Industrial service	13.4	13.2	9.9
Public fire protection service	22.5	26.9	28.0
Miscellaneous municipal sales	1.6	2.3	2.2
Other revenue	1.0	0.8	0.7

major classes of service. This we have done for Class A and B utilities only. This includes a group of 36 municipally owned and 7 privately owned plants, or a total of 43. These are the plants reporting a complete breakdown of revenues by classes of service. We have also excluded from Class A and B utilities certain predominantly residential suburbs for the reason that the inclusion of these utilities would tend to distort the percentage figures.

The figures appear in table 7. On the average, the 43 water plants included in this part of the study receive about 45 percent of their revenues from residential service, from 12 to 15 percent of their revenues from commercial service, from 10 to 13.5 percent from industrial service and from 22.5 to 28 percent from public fire protection service. These figures neglect certain incidental and minor sources of revenue. In general, the larger plants show, as might be expected, a somewhat higher proportion of their revenues received from industrial service. The most striking exception to this statement is in the case of privately owned plants in the large-community-size group where the proportion of revenues from industrial service is lower than in any other size or ownership grouping.

TABLE 8

Public fire protection service, 1934

	POPULATION GROUP			
	20,000–68,000	8,000–20,000	4,000–8,000	3,000–4,000
Per dollar revenue, **percent**				
Municipal	22.4	25.3	28.6	27.4
Private	23.7	29.5	49.5	30.6
Municipal and private	22.6	27.5	29.6	27.6
Per capita, **dollars**				
Municipal	1.10	1.35	1.48	1.44
Private	1.37	2.17	3.58	1.72
Municipal and private	1.13	1.57	1.58	1.46

Because of the interest in methods of charging for public fire protection service we have made an additional compilation of the average proportion of gross revenues received from this class of service. In this comparison we have taken all municipally owned and privately owned plants included in this study for the year 1934. (For summary see table 8.) On a per dollar of revenue basis as indicated above, from about 22.5 to 29.5 percent of gross revenue is received for public fire protection service. On a per capita basis for both municipal and privately owned plants public fire protection service charges average from $1.13 per capita to $1.58 per capita in the different size groups of plants. In general, the charges for public fire protection service on a per capita basis are higher for privately owned plants than for publicly owned plants. The larger

plants show a somewhat lower per capita figure than do the smaller plants.

These figures speak for themselves. They are no more accurate than the reports of the utilities to the Commission. Such conclusions as can be drawn from a first study of the figures have been indicated.

ACKNOWLEDGMENT

Grateful acknowledgment is given for the ideas and assistance of E. W. Morehouse, Chief of the Rates and Research Division of the Public Service Commission of Wisconsin.

(Presented before the Wisconsin Section meeting, November 5, 1935.)

SPECIFICATIONS FOR WATER MAIN CONSTRUCTION

By Albert R. Davis

(*Superintendent, Water Department, Austin, Texas*)

A set of specifications for water main construction would not differ materially from any other type of construction, except as to details. Every set should include the following items: Index, Advertisement, Notice and Instructions to Bidders, (Labor Classification and Minimum Wage Scale, optional), Bond, Labor Bond, Financial Statement, Experience Record, Equipment Schedule, Standard Form of Agreement, Proposal, General Conditions of Agreement, Detailed Specifications, and Plans. Then to make it look ship-shape, it should be bound in a neat leather folder with gold lettering on the back. All of the above items have their respective duties to perform, and they are necessary, but only the last four items will be discussed and in the reverse order named.

PLANS

Every job in its early stages is in the mind of the engineer, and he must find some way to express his thoughts clearly and concisely. It has been said that necessity was the mother of invention, and since there never was an engineer that liked to talk, he had to invent some method of making paper talk for him. As a result we have tracing paper and ink, and erasers. Plans for water lines are not very difficult to produce if the necessary information is available. The first requirement is to secure a plot and map of the area where the line is to be laid. If in city streets or alleys, then the width and length should show by lettered dimension and scale. The scale used is not important; however, it is well to have it large enough to be easily legible for convenient use in the field. On the plan, the location of the proposed water main should show by a solid or dashed line whichever is preferred. The distance of the main from visible land marks is important. This is necessary for the contractor in inspecting the job to be able to locate the required position. The location for a water main should be furnished by the city engineer. It is a funda-

mental function of the city engineer's office to have a record of all existing underground structures in streets and alleys, and from his record he can determine the location and grade of the proposed water main. The city plan engineer should also be consulted as to probable changes in existing streets and alleys. The plans should also show the location of all special castings, valves, fire hydrants, taps, meters, or what have you. Where manholes are required around valves, complete plans should be included, showing size, wall thickness, top thickness, location of reinforcing steel, and a schedule of reinforcing steel. If road boxes are to be used, a drawing should also be included in the plans. The profile which accompanies the plan should show the elevation of existing utilities, other underground structures that are to be avoided during the construction of the main, and depths necessary for the contractor to go to complete the work on which he bids. The profile should show the depth of the main by actual elevation, and by scale and measurement from the normal surface of the ground. In short the plan is a picture of the work to be done.

The sizes of the main and its accouterments are the first problems of the engineer. This, of course, is based on the needs of the particular locality, and can be arrived at by a proper study of domestic and fire demands plus an allowance for growth and development.

DETAILED SPECIFICATIONS

Now comes the necessity of taking the set of plans, which anyone ought to be able to take and do the job, and writing out in words exactly how, when, and why the work is to be done. First, the specifications for materials to be used should be written so that the proper materials will be used in the construction of the job. Competition of reliable material companies is desirable; therefore, the specifications must not be closed to them. Sufficient strictness to keep out fly-by-nighters is recommended. The Federal Government has published a standard specification for cast iron pipe in both the bell and spigot type and the bolted type. These bulletins are numbered, and may be referred to in specifications by number, thus eliminating lengthy wording for pipe. Special castings, fire hydrants, valves, and meters, if made to meet the requirements of the A. W. W. A., will be found satisfactory, and the engineer will have little trouble in selecting a good reliable company to furnish his requirements. Of course, these items must have certain city standards. Brass goods, joint materials, road boxes, manhole covers, etc., are not covered by

any standard set of specifications, and these details must be given by the engineer.

Every engineer has his own ideas as to wording of certain thoughts which he desires incorporated in the work. This is quite proper as long as his wording conveys clearly and concisely what the contractor is expected to do.

In a recent set of specifications, prepared by the writer for the City of Austin, covering 20,000 feet of 20-inch and 8,000 feet of 12-inch, a total of thirteen pages, single-spaced, was required to cover the necessary phases of detailed specifications. Included in these thirteen pages will be found the information concerning the work to be done, an explanation of alternate bids, material specifications, construction; and under the heading of "Construction," such items as testing, trenching, classification of materials excavated, computation of rock excavation, pipe laying, procedure in making joints, setting of valves, fire hydrants, placing of special castings, goosenecks, manholes and road boxes, traffic, protection of public utilities, testing for leakage, backfill, sterilization, wet connections, barricades and lights, replacement of street surfaces, measurements, sterilization of jute and hemp, payment for water used, clean-up, and basis of payment. There are a few of these items that might very properly be discussed at this time. First is the matter of classification of materials excavated. Our experience in the City of Austin has indicated that there are two kinds of excavation—one is earth and the other is rock. This classification eliminates many tedious arguments with the contractor.

Another item is taking care of traffic during the progress of the work. A contractor, as a rule, working in the City is not particularly concerned about keeping street crossings open so that the public may have use of the streets as much as possible. A paragraph in a set of specifications outlining what may or may not be done will usually be found advantageous, especially to the owner, or, in most cases, the city water department.

TESTING FOR LEAKAGE

The standard requirement for leakage is 200 gallons per inch diameter of pipe per mile per day. It occurs to me that this leakage is too liberal; and if a line is properly laid and the joints properly poured, a better leakage allowance would be 20 gallons per inch per mile per day.

Sterilization of the new pipe after it has been placed in the ground is very important. There are two accepted methods, and possibly others, of securing thorough sterilization of the inside of the pipe. One method is by injecting free chlorine into the water filled main, and the other is to insert at various intervals as the line is laid a reagent containing a high percentage of chlorine, which is liberated when water is applied. My experience has been that the use of chlorine from drums is the preferable method.

The following paragraph on measurements is quoted from a recent set of specifications:

Measurements. The pipe line measurements will commence at the bell of the existing connection, and shall extend to the bell of the existing connection where section of pipe is completed. No deductions will be made for the length of any interlying special casting, gate valve, or other fitting. This specification is being written for the purpose of informing the contractor that no meticulous measurements involving the above mentioned fittings will be made.

PROPOSALS

The proposal in the set of specifications is based on the Instructions to Bidders and presupposes familiarity with all of the other items included in a set of plans and specifications, and is so set up that the contractor may submit a price or a bid on the work required to be done. The proposals for water main construction are usually made up so that a different price can be bid for pipe, etc., of different sizes at different prices per foot or per unit. The prices should be submitted in writing and in figures, and in case of a difference between the figures and the writing, the amount specified in writing should be the one to govern. The proposal should also provide for the total price for the job complete in dollars and cents. It is important that the unit prices be taken in order to establish a price for a particular kind of work; and if during the course of construction of the work, it is deemed necessary to add to or take from the original amount specified, the bid price per unit will govern in making such additions or deductions. It has been found quite convenient, especially at lettings, to have the sheet or sheets upon which the proposals are entered to be of a different color of paper from the rest of the specifications. While this is not essential, it has been found to be useful. In setting up the proposals for a water line, the engineer must be governed by the method he desires to pursue; that is, if the contractor is to furnish all material, and do all work, the proposals must so state; or if the owner

desires to purchase the material separately from the material companies, and have the contractor bid on the installation of the work, the proposal should also state definitely and clearly exactly what the contractor is bidding upon.

During the recent months, this country has seen quite a number of PWA water works lettings, and from the best information available, the contractor has been required to furnish materials and do the work upon which he bids. This method is preferred to separate contracts for each material required, and a contract with the bidder doing the work. Where separate contracts are made, the owner will find himself faced with drawing up several contracts in legal form with the various material companies and one with the contractor. The other method requires only one contract, and that is with the contractor who bids the work complete.

GENERAL CONDITIONS OF AGREEMENT

The American Society of Civil Engineers has published a General Conditions of Agreement, and the last revision was April 15, 1932. This General Conditions of Agreement was worked out by a group of engineers, contractors, and materialmen, and is the result of several years' work by committees appointed by the different groups. Looking at it from the engineers' viewpoint, there are certain facts which can be left out, but when considering it from the standpoint of the contractor and the materialmen, it seems that the form as developed by the Association is fair to all concerned. There are forty-six different articles in this document, and they cover almost any contingency that might arise. The Standard Form of Agreement was also produced by the American Society of Civil Engineers as of October 28, 1921.)

(Presented before the Southwest Section Meeting, October 14, 1935.)

DEEP WELL PUMPING

By C. N. Ward

(Chief Engineer, Mead, Ward and Hunt, Consulting Engineers, Madison, Wis.)

A general consideration of deep well pumping involves too many factors to permit even a complete outline of all of them in a limited paper such as this. Aside from the diversity of natural factors to be met, there are others introduced because of local conditions such as limited funds which prohibit the installation of methods or equipment to supersede existing ones which are obsolete or which originally were improperly selected. This paper will present a few observations of the writer on the general recent trend of developments in this field and of some particular experiences which may be of interest.

BUCKET PUMPS

The reciprocating or bucket type of pump has served for a great many years as a practicable and effective machine for lifting water from deep wells. It is used today for private water supplies and in many small communities not only to raise water from the well to ground level, but also to deliver it against pressures carried in distribution systems. The writer has tested several old pumps of this type during the past few years and has found that they generally give efficiencies which are surprisingly high considering the conditions under which they were installed and the maintenance given them. In one or two instances the efficiencies are probably higher than could be secured with other available types which could be installed in the existing wells.

A deep well electrically driven bucket pump installed in 1897 which served a village in central Wisconsin continuously gave a wire to water efficiency of about 60 percent when tested in 1934. Small and crooked bore holes of wells often prevent the proper setting of bucket pumps and, in such cases, frequent pulling of the pump is necessary to replace pump rods. Operators of pumps which are thus improperly installed should not be too critical of this type of

pump because of high maintenance costs. In many cases more modern types of pumps could not be installed at all in the existing wells and if installed would also give considerable trouble in operation.

The manufacturers of the bucket type of pumps have sought to increase their rates of delivery by perfecting the mechanism of pump heads so as to give reciprocating motions that will permit higher speed of bucket travel. This has been accomplished by developing motions that decrease the accelerations at the lower end of the stroke when stresses of pump rods are the greatest and to speed up the travel in the middle of the stroke. Many of these developments are interesting but some types have proved very impractical under operating conditions.

AIR LIFT PUMPS

Air lift pumps were used to a considerable extent in municipally owned wells in this State until a few years ago. The general trend recently has been away from this type of pump. For those who by chance may not be familiar with this pump, the following brief description is presented. An air lift pump has no valves or moving parts below ground. It consists of a pipe, called an "eduction pipe," which extends down into the well considerably below the lowest level to which water is drawn in the well. A small pipe is placed in the well to the bottom of the eduction pipe to carry compressed air down to that level. The arrangement or fitting at the bottom of the eduction pipe where air is admitted to the water is termed the "foot piece." In operation, air is forced into the bottom of the eduction pipe where it rises in the water in the form of bubbles of various sizes. The density of the mixture of air and water is less than that of water, and the column of the mixture rises above the pressure level in the well. As more air is added the column of air and water rises to the top of the well, and a flow upward results.

When an air lift pump is improperly designed or operated it can be extremely wasteful of energy. In some cases the air supply furnished is inadequate in amount and the mixture of air and water is held in the eduction pipe at a level below the point of discharge, and only occasionally does it rise high enough to permit flow. At the other extreme, the amount of air supplied is so great that very little water gets into the eduction pipe, and the discharge at the top of the eduction pipe is mostly air.

These pumps usually have been designed by rule of thumb meth-

ods and the results secured were more or less of a gamble. A large part of the uncertainty of performance has been eliminated by methods of design which have been developed in recent years. It is possible to design an air lift pump to meet particular operating conditions with the same certainty that one has in designing many other hydraulic devices. Air lift installations which may have been satisfactory in the beginning have become extremely inefficient because of changes which have occurred such as a lowering of ground water levels or a decrease in the yield of the well due to other causes.

An air lift pump at its best cannot meet the maximum efficiencies now attainable with other types of pumps. The highest overall efficiency, wire to water, which the writer has observed in tests is about 40 percent. With the development of other types of pumps for deep wells with greater capacity and reliability, the relative usefulness of air lift pumps has decreased.

The greatest advantage which an air lift pump has is dependability. All moving parts are readily accessible for maintenance. Should a breakdown occur the trouble can quickly be determined and repairs started at once without removing heavy pipe and equipment from the well.

Air lift pumps operate satisfactorily in wells which develop considerable sand. They also have an advantage over other pumps where aeration of the water is beneficial.

TURBINE PUMPS

The greatest change in deep well pumping has come during the last two decades with the development and perfection of the so-called turbine pump. This type of pump is really a centrifugal pump which has been designed to go into the relatively small space which is available in deep wells. Turbine pumps are of comparatively recent development. The first one used in the United States was built by the Byron-Jackson Company for the Pabst Brewing Company of Milwaukee, and was selected and installed under the direction of Dr. Daniel W. Mead in 1903.

The reliability and the efficiency of these pumps have been greatly enhanced by developments in the past years. Thrust bearings which carry the weight of the shaft, impeller, and thrust of the water have been so perfected that they give little trouble in pumps which are properly installed in wells that are adapted to this type of pumping.

Turbine pumps are now available for operation in practically

any well which is likely to be encountered. Overall efficiencies have been increased over a considerable range of operating conditions due to the advance in the hydraulic turbine design of the pumps. The turbine pump has practically displaced all other types of deep well pumps excepting those for very small rates of delivery.

An acceptance test was run by the writer about three years ago on a deep well pump which was guaranteed to deliver 2700 gallons per minute against a total head of 225 feet with a wire to water efficiency of 68.5 percent. The writer and many others were very skeptical about the attainment of such a high efficiency. Accurate and carefully made tests disclosed a wire to water efficiency of 69.9 percent. Unfortunately there was a considerable amount of sand carried in the water from the well which caused a relatively rapid deterioration of the pump. It was stated that the efficiency of the pump dropped very materially within a few months as disclosed by the daily operating records.

The type of shaft bearings and methods of lubrication to be selected are important and difficult problems. There is a difference of opinion among pump designers as to the relative merits of oil lubricated metal bearings and water lubricated rubber bearings. Some pumps are designed in such a manner that there is little assurance that so-called oil lubricated bearings in the lower part of the columns actually receive any oil because of the difficulty of keeping water out of the lower end of the oil pipe. In certain oil lubricated pumps no provision is made for oil lubrication of bearings between the various bowls and the end bearing. With an oil pressure system which reasonably assures that all oil bearings are filled with oil, there is the possibility that excess oil will be forced out to mix with the water. These difficulties are eliminated with water lubricated rubber bearings but new problems arise in their use. A shaft enclosing pipe which carries clear water is necessary where silt and sand are carried in the water to be pumped. If the static water level stands at any great depth below the pump head, it is probably desirable to provide facilities for admitting water to the shaft bearings from some outside source during the period when the water level is being raised in the pump column during starting. That the use of water lubricated bearings has not met entirely all operating conditions is evidenced by the fact that certain manufacturers have abandoned designs of water lubricated bearings which they developed and are now using another type.

The writer has had but one pump with water lubrication of shaft bearings installed under his direction and that was during the past year. The period of operation of this pump is too short to permit a decision on the merits of this installation. Generally, pumps with oil lubricated shaft bearings, which have been selected and installed under the direction of the writer or have come under his observation, have given good service. There have been just enough failures of shafting due to poor design of bearings, faulty lubrication, and improper setting of the pumps to indicate the need of special attention to this phase of the problem in selecting new pumps.

SELECTING DEEP WELL PUMPS

When a new well is constructed it is comparatively easy to get necessary data to determine the yield which can reasonably be developed and the corresponding drawdown in the well. A mistake which is commonly made is that of discontinuing pumping tests too early. Very often observers are misled in thinking that the final drawdown for a given rate of pumping has been secured long before stable conditions are reached. In some instances a well in an area which has not previously been drilled will indicate an unusually high specific yield for a few hours, and possibly after ten or more hours there may be a substantial progressive increase of the drawdown. Possibly in such cases an upper water bearing stratum is being relieved of water which has been collecting there over a long period of time and after it has once been drained down it does not contribute in a substantial way to the well. Observations taken in the early stages of test pumping are in such cases very apt to be misleading.

It is important to know what the static water level is, that is, the level to which water stands in the well before pumping starts. Since there is a seasonal variation in static water levels in many localities, one should determine from existing wells if possible the range of such fluctuations and the relative level which exists at the time of the test of the well in question. Even the static pressure level in the deep seated sandstones of Wisconsin in some places shows quite a variation between periods of ample rainfall and of droughts.

For a deep well driven into artesian water bearing strata it is usual to find an approximate straight line relationship between the drawdown, that is, the distance the water is lowered below static level, and the rate of pumping. In other words, the drawdown is

approximately directly proportional to the rate of pumping. The specific yield is the rate of pumping generally expressed in gallons per minute per foot of drawdown. Where wells are pumped at extremely high rates it is often found that there is a departure from the straight line relationship above mentioned and that the specific yield decreases.

A convenient way to study the relative merits of various pumps for the well in question is to plot a curve between drawdown and rate of pumping as determined in tests of the well. To each ordinate of this curve add the vertical distance from static level to the pressure level against which the pump will operate plus friction losses in piping and pump columns for each rate of pumping. The resulting curve is a "head discharge" curve which can be plotted upon the characteristic curves of the pump as furnished by the manufacturers. The rate of pumpage, efficiency, and power to be delivered to that pump can be read from the characteristic curves on the ordinate which passes through the point of intersection of the "head-discharge" curve as above determined with that of the "head discharge" curve of the pump.

Where two pumps show the same efficiency when applying the above test, one would favor the one that would give the higher efficiency should there be a decrease in the specific yield of the well. Generally, the writer prefers a low speed pump. With such a long shaft between the motor and the pump, the problem of lubrication and vibration is much simpler, maintenance is less costly, and the life of the pipe is longer where the speed of operation is less. New wells should be drilled with an adequate bore to permit the installation of a pump which will furnish the required lift with as few stages as practicable when operating at a comparatively low speed.

The bore and alignment of old wells should be carefully checked so that the size of pump which can be installed can be definitely specified. Rather unfortunate and complicated conditions arose in one instance where a pump was provided which could not be installed in the well.

If there is a substantial lift to be encountered above the discharge opening of a turbine pump, it is desirable to provide a booster pump in almost all cases. A reduction of the load upon inaccessible parts such as the rotors of a turbine pump is, in the opinion of the writer, the proper procedure. Higher overall efficiency can usually be ob-

tained with booster pumps and, in many cases, the initial cost of the installation is not increased when a booster pump is used.

A low service reservoir between the turbine pump and the booster pump is often a desirable feature. Where sufficient volume of storage is provided, the deep well pump is not required to start and stop often, as is the case in many distribution systems without adequate elevated storage and where turbine pumps deliver directly to the system either with or without a booster. Where even a small amount of sand is pumped from a well, it is desirable to have a low service reservoir so that the sand can be settled out and not be delivered into the distribution system.

In all cases, provision should be made for determining the water level in the well after the pump is installed. A small pipe, called a "tell-tale pipe" is usually used for such purpose. Many dollars have been wasted because of the impracticability of determining water levels in wells because no provision was made for that observation. Without a knowledge of the drawdown during operation, one cannot tell whether a decreased rate of pumping is due to deterioration of the pump or to the well. An operating record which omits data on the drawdown in the well is apt to be of limited value.

A tell-tale pipe should be installed with its lower end within a foot or two of the end of the drop-pipe of a turbine pump or from the footpiece of an air lift pump. In one installation a tell-tale pipe was extended down somewhat below the level at a maximum drawdown on a well fitted with an air lift pump. The footpiece was located nearly 200 feet below this level. A good water bearing stratum existed between the end of the tell-tale and the footpiece, and there was considerable flow of water down the bore of the well and outside the eduction pipe. The resulting hydraulic friction was from 12 to 15 feet, and of course, the true pressure level from which the pump was delivering water was from 12 to 15 feet lower than the level of the water in the well as disclosed by the tell-tale pipe. When the pump was credited with the true lift under which it operated it was found that it gave as high an efficiency as could be expected.

Care must be taken in using a tell-tale pipe in a well which is sealed at the top and where a vacuum exists to determine the actual water level in the well and to calculate the true head delivered by a pump from such a well. It is necessary to have a separate gage connection to the top of the sealed space so that the pressure within the casing can be measured.

CONCLUSIONS

There are essentially three types of pumps which are used in deep well pumping, and the most important one from the standpoint of use for municipal water supply systems is the turbine pump.

A great deal of attention must be given to the selection of a turbine pump because both its efficiency and the rate of delivery change with the head to be pumped against.

Booster pumps, preferably those taking water from a low service reservoir, are recommended to operate in conjunction with deep well turbine pumps rather than booster pumps connected directly to deep well pumps.

Air lift pumps still have a useful place under certain conditions. Where reliability and continuity of service are the prime factors or where sand is encountered to such an extent as to cause rapid deterioration of other types of pumps the air lift still can be used to advantage. Bucket pumps give very good efficiencies and their use undoubtedly will be continued for a long time in small wells for furnishing water for small distribution systems.

(Presented before the Wisconsin Section meeting, November 5, 1935.)

METER SHOP PRACTICE

By M. B. Cunningham

(Assistant Superintendent, Water Department, Oklahoma City, Okla.)

In discussing meter shop practice, we are to give consideration to three phases of meter work upon which the success or failure of the meter shop will depend. These phases are:

1. Keeping the meter out of the shop.
2. When to take the meter to repair shop.
3. What to do with the meter in the shop.

REPAIR PREVENTION

We hear a great deal of talk about fire prevention and of course realize its importance in preventing fires. Why should not the water department give some thought to meter repair prevention as well as to the repairs themselves. By this I mean that a meter shop should set up certain standards of practice and eliminate haphazard methods. An effort to keep the water meters out of the repair shop is certainly worthy of careful consideration.

The time to start this practice is in the wise selection of the most useful and dependable water meter adapted to local water condition and service requirements. Too often the purchase of water meters is based solely on price or salesmanship.

After all a water meter is asked to perform almost a miracle inasmuch as it will be set on a service and be left to fulfill many requirements automatically, almost without attention, for every hour of its useful life. This meter will encounter inside and outside forces which it must resist, such as extreme heat and cold, dirt, water, wear and tear, excessive water pressure and low pressure, corrosion, and rates of flow above and below its designed limits. Yet this meter is expected to record accurately every gallon of water used by the consumer and give uninterrupted service. For this faithful service the maintenance and depreciation must not be too high.

It is not unreasonable to expect this service provided the proper stress has been placed upon the necessity of selecting a meter by its ability to perform these duties.

To be able to make this selection requires close coöperation between the water department and the meter manufacturers. My observation and experience convinces me that most meter manufacturers are anxious to coöperate to the fullest extent. This is especially true if the water department has actual facts and figures based on meter performance.

To obtain accurate meter performance records the water department must have more than a repair shop. There must be intelligent observation of meters, laboratory testing and analysis, various records kept accurately and continuously. Such a system leads to results worth all the time and effort required to put it in force.

These records and observations make it possible to take a new meter and judge its good and bad points. The record made by a meter in service is the best method to judge its merits.

In Oklahoma City when new meters are to be purchased the meter manufacturer is required to submit a sample meter before awarding bids. (Sample meter is required on $\frac{5}{8}$- to 2-inch size only.) Specifications require that meter furnished shall conform with cold water meter specifications as recommended by the American Water Works Association, $\frac{5}{8}$- to 2-inch Disc, 2- to 6-inch current compound.

In selecting a meter for Oklahoma City, special consideration is given to the following points:

1. Meter must be best quality model or type made by manufacturer, as bid on second grade meter is not considered.
2. Accurate registration on minimum, average and maximum flows as required by A. W. W. A. specifications.
3. Precise and uniform workmanship throughout meter.
4. Proven design.
5. Proper use of metals to minimize corrosion.
6. Adaptability to local water condition such as resistance to corrosive water and resistance to deposits from water treatments.
7. Adaptability to local weather.
8. Resistance to hot water.
9. The ability to give uninterrupted service over a long period of time with low maintenance and depreciation costs.
10. Detail study made of meter to determine its expected performance in actual service. Observations and records kept on like meters already in service is used largely as the basis of conclusion.

11. If trouble has developed in like meters already in service, meter representative must furnish proof that trouble will be corrected in meters to be furnished.
12. The willingness to pay manufacturers a just price for a water meter meeting local requirements.

SELECTING CORRECT SIZE AND TYPE METER FOR CONSUMER'S SERVICE

Selecting the correct type of meter for each consumer's particular service is another very important practice to keep the meter out of the repair shop.

Water meters are made in various sizes and designs, each to fulfill a specific purpose.

The water department desires to sell all the water a consumer wants. Also the meter is expected to register all water used by consumer. To obtain this registration correct meter size and type must be used. The water department must have information concerning minimum and maximum rates of flow; for 1½-inch and larger meters it is sometimes necessary to inspect consumer's premises or building to get this information. Maximum useful life of meter depends largely upon its use on rates of flow within limits of meter design.

It is not efficient to use a 2-inch meter where a ¾-inch would be better, or a 2-inch in place of a 3-inch. In other words, if a meter is designed for a maximum rate of flow of 160 and the actual demand is 200 g.p.m. then this overwork will shorten the life of meter. The meter will be brought to the shop worn out.

METER HOUSING

After the meter is purchased, correct installation will help to keep the meter out of the repair shop.

Meter housing must be properly designed to protect meter from local weather conditions and tampering from outside sources.

In Oklahoma City the frost line is seldom more than 18 inches. For ⅝- and ¾-inch meters a standardized meter tile is used. This tile is made up of a cast iron ring 18 inches in diameter with a 14-inch cast iron lock type lid. The body of tile is made of 1½-inch concrete with 12 gauge wire reinforcement. Inside diameter at bottom, 20 inches. Depth 18 inches. This tile sits upon a 6-inch concrete tile ring extension giving a depth of 24 inches over all. The meter is set 6 inches off bottom.

A slightly larger tile is used for 1- or 2, ⅝-inch meters. 1½-, 2- and 3-inch meters or a multiple of smaller size meters are housed in

TABLE 1

SIZE, INCHES	NORMAL TEST FLOW LIMITS REGISTRATION RECORDED NOT LESS THAN 98, NOT MORE THAN 102 PERCENT OF WATER ACTUALLY PASSING THROUGH METER,* GALLONS PER MINUTE
$\frac{5}{8}$	1 to 20
$\frac{3}{4}$	2 to 34
1	3 to 53
$1\frac{1}{2}$	5 to 100
2	8 to 160
3	16 to 315
4	28 to 500
6	48 to 1000

*** For size meter consult meter manufacturer for recommended rates of flow for current type meters.**

TABLE 2

SIZE, INCHES	MINIMUM TEST FLOWS REGISTRATION RECORDED NOT LESS THAN 90 PERCENT OF WATER PASSING THROUGH METER, GALLONS PER MINUTE
$\frac{5}{8}$	$\frac{1}{4}$
$\frac{3}{4}$	$\frac{1}{2}$
1	$\frac{3}{4}$
2	2
2 compound	$\frac{1}{4}$
3	4
3 compound	$\frac{1}{2}$
4	7
4 compound	$\frac{1}{2}$
6	12
6 compound	$\frac{3}{4}$

TABLE 3

RATE OF FLOW AT WHICH METERS WERE TESTED, GALLONS PER MINUTE	AVERAGE PERCENT OF ACCURACY	PERCENT OF WATER PASSING THROUGH METER AT THESE RATES OF FLOW	PERCENT OF WATER RECORDED
$\frac{1}{4}$	73	15*	10.95
$\frac{1}{2}$	95.6	10	9.56
5	100.2	50	50.10
10	101	20	20.20
15	99.8	5	4.99
Total..................		100	95.80

*** Estimates run as high as 30 percent at times.**

a concrete box precast in four sections with cast iron removable frame with 14-inch lock lid. 4-inch and larger meters are housed in a box built up of tile or brick with cast iron frame and lid. A lock lid on every meter box is good protection insurance. It prevents tampering with meter seals, and register; it reduces broken meter glass and loose hands. Complaints of consumers are also reduced because meter is most likely to be in good working order. There should be no guess work on the larger size meters. Rates of flow meters are now available so any size meter can be checked continuously on 24 hour charts. If this equipment is not available the peak flow can be checked by a stop watch with very good results.

The over working of a meter should not be tolerated, neither should the minimum flow or under registration be neglected.

Minimum and maximum rates of flow are shown in table 1.

Table 2 shows minimum flow required for 90 percent registration. Rates of flows less than these amounts will register on meter from 0 to 90 percent. A 2-inch meter for example is required to register not less than 90 percent of water passing through the meter at the rate of 2 g.p.m. If this same meter is compounded with a $\frac{5}{8}$-inch small flow meter it is possible to get better than 90 percent registration above $\frac{1}{4}$ g.p.m.

Table 3 shows 24 hour records of rates of flow recorded on 15 meters used as basis for these percentages. Ten gallon tests were run on each rate of flow. Percent of accuracy was determined by weighing water. Meters were in service 2 to 7 years.

Table 3 shows clearly the extreme need for meters that will register on small rates of flow.

WHEN TO BRING WATER METER TO REPAIR SHOP

The ideal time to remove a meter from service is before it begins to pass water which does not register. This brings up the question, how can this be accomplished. Conditions vary so widely I am convinced you will have to work out your own solution. We can all agree that a meter should be removed from service before registration fails entirely.

In Oklahoma City a definite record system has been set up so we can establish within a reasonable degree of accuracy the useful life of each size and model meter.

For example, several years ago we found that one group of $\frac{5}{8}$-inch meters had a poor mixture of metal in a certain part. Records in the repair shop show galvanic corrosion had caused a drastic reduc-

tion in registration in 2 years. All meters of this size and model were brought to the shop immediately for correction of parts.

On the other hand, other extremes are found where $\frac{5}{8}$-inch meters can safely stay out 7 to 8 years and show very good average registration on both minimum flow of $\frac{1}{4}$ g.p.m. and over. All meters coming into the shop should be tested and records made showing percent of accuracy on rates of flows shown in table 1 for normal flow tests and minimum flows shown in table 2. This record can be recorded on the meter repair ticket. In addition to flow test records for incoming meters, a record should also be made showing:

Size
Make
Model
Total reading while in service
Number of years in service
Comments on what was cause of repairs
Labor and material cost

If these results are tabulated each month for the individual meter or by size, make and model as a group representing like meters, definite conclusions can be drawn.

If you want to tackle a very interesting problem just make up some forms based on your local needs and have your shop begin to fill them out.

These records properly executed and tabulated will tell the story of when to bring water meters to the shop.

WHAT TO DO WITH METER IN REPAIR SHOP

Minimum flow tests. I like to think of a meter being rebuilt rather than repaired. A properly rebuilt meter should be as good as a new meter, not as far as outside looks go but mechanically sound and free from defects. Percent of registration on minimum, average and maximum flows should be required to equal that of a new meter. One of the most neglected tests given a rebuilt meter and yet the most necessary to check the workmanship is the requirement of 90 percent registration at minimum rates of flow. For example a $\frac{5}{8}$-inch meter should test 90 percent or better on $\frac{1}{2}$ g.p.m. This test is very important as 15 to 30 percent of the water used on the $\frac{5}{8}$-inch meter is used at this rate of flow.

The necessity for minimum registration on meters was discussed about five years ago at the Southwest Water Works Convention.

For some time I gave considerable thought to the testing of all rebuilt meters on this flow. After giving it a trial I immediately saw its necessity. A Mueller multiple meter testing attachment was purchased for our Mueller testing equipment. Every meter rebuilt for the past five years has been given this test. Table 2 shows the minimum flow in g.p.m. and the required percent of registration. This is the most important test given a meter. It does more to require good meter shop practice than anything I know. The average increase in cost for meter repair since requiring this test has been 30 cents each. Most of this cost is labor. This increased cost represents a cost per meter in service per year of 3 cents. The average ⅝-inch meter in Oklahoma City remains in service 10.7 years before it dies. Seven years is the average safe registration time limit established on domestic meters. One and one-half thousand gallons of water costs the consumer 30 cents. If a ⅝-inch meter only picks up 1½ m. gallons of water in seven years the department has lost nothing. It will do more than this, for in Oklahoma City we have gained close to 5 percent in metered water sold compared to the pumpage and a large part of this gain is due to better meter shop practice.

Good meter shop practice demands special equipment. No meter shop is complete without gauges. These are used to determine the wear on disc balls and disc sockets, also to detect warped discs when not visible to the eye. A piston wears unevenly, as do the bottom and the top plate on piston chamber. All of these surfaces can be "trued" on a sanding disc or lathe. If this is done a visible reading micrometer is a time saver. If a piston surface is "trued" the piston ring surface must necessarily be cut a like amount, leaving about two thousandths of an inch clearance. To loosen the bearings in a tight register we use ½- by 4-inch fairly hard rubber wheel substituted in the place of a grinding wheel hand operated. I do not want to leave the impression however that this will take the place of taking register apart and properly cleaning.

Safety goggles should be used by workmen cleaning meters. Grinding wheels should be covered by safety glass guards. Safe methods must be used because protection to the eyes is most necessary.

Repairing meter parts. No meter should go through the shop without being torn down and having each part examined, replaced, or rebuilt if worn beyond safe limit. Several details may be of interest to you.

The register box lid should be loose enough to fall in place of its

own weight. If it is too tight it will break off leaving the glass exposed.

Meter glass, in my opinion, is one of the weak parts of a water meter. A broken glass admits dirt and water on the register, which must be protected. Pieces of broken glass left on the dial result in loose meter hands, which causes no end of grief in determining the correct water consumption. I hope some one will develop a non-breakable glass for use in water meters.

Meter registers can be protected around the outside edge by buying what is known as a dust ring. This ring is inexpensive and keeps dirt from collecting around sensitive bearings.

A close inspection of register gear teeth must be made. It is not uncommon to find several teeth broken. Individual gears should be stocked for such replacements. Rubber bushed registers have proved best in Oklahoma City.

All register hands are filed to a clean surface and resoldered. Electric soldering iron is used with resin core hard solder. Acid core solder should not be used as it will cause parts to turn green. This corrosion causes tight bearings. We set all meters to read 0.

Stuffing box. The stuffing box shaft should be examined carefully because excessive wear may require replacement. New packing should be used by all means. We have found sheet cork with leather washer on bottom and top to be the best. We have on trial a number of meters using a plastic composition. Rawhide packing, which we tried, had to be replaced after three years use.

Oil enclosed gear trains are required in new meters. We have experienced trouble in some of the first designs used to replace open gear trains. Careful examination is given gear trains needing repairs. Most of these are rebuilt with new improved parts, or else new and improved gear trains are purchased for replacements. Special grease is obtained from the various meter manufacturers. We still have about eight thousand meters with open trains. These are gradually being replaced with oil enclosed trains when the meters come through the shop.

Parts are stocked and any part can be replaced, whether open or oil enclosed. All trains are cleaned and every part examined thoroughly before being replaced in meter. A little loose play in gear bearing sometimes causes gears to bind. I examined a meter several days ago where a gear had three out of nine teeth sheared off. This would not cause a complete loss of registration, but would cause a lack of registration.

Disc and disc chamber. A meter disc wears most on upper and lower ball surfaces. If the disc is of the cupped type, replacement should be ordered giving in thousands the oversize wanted to take up wear in disc chamber sockets.

A flat disc can be obtained in three parts. This permits the use of a gasket beneath balls which compensates for the wear in the sockets.

Flat discs can also be ordered oversize. Extreme care must be taken to examine disc for warping. Most warped discs are easily detected, but a slightly warped disc is hard to detect and will not pass minimum flow test with sufficient accuracy.

Current type meters as a rule are so constructed that working parts can be examined in service. Inspections on these meters should be made while meter is in service just as often as each meter indicates this need. Bearings need adjusting. If compounding valves with small flow meter is used, compound valves may need checking as to freedom of operation. Small flow meters should be run not to exceed a small flow, while the large meter is in use. This saves excessive wear on small meters which are used only to obtain proper registration when the large meter is not in use.

Corrosion in meters. The Water Works Practice Manual explains the three most common types of corrosion. Self-corrosion, galvanic corrosion and electrolytic corrosion. Destructive forces such as these are known to cause much of meter parts failure. This is where a meter laboratory comes in handy. Your local water condition will have much to do with the extent of corrosion in metals used in meters. Someone in the department must be able to work out the best solution and make recommendations as to corrective materials to replace those causing trouble.

As an example in Oklahoma City a meter comes to the shop for repairs. Examination shows a white deposit of zinc salts on some part of the meter such as screws, division plate in disc chamber, bottom liner or various other parts. Examination of the metal used in these parts when new would show an alloy of copper and zinc called yellow brass. Yellow brass has a copper content of 65 to 80 percent, zinc from 35 to 20 percent. When metal is exposed to water there is a minute flow of current set up traveling from zinc to copper. The zinc goes into solution leaving the copper soft and crumbles easily. Meters properly re-built must have these affected parts replaced with corrected metals.

Replacements are made with bronze or red brass. Bronze is an alloy of copper and tin. Red brass contains not less than 85 percent

copper, 15 percent zinc and tin. Monel metal has been used with good results, but costs more than bronze or red brass.

It is the old meter purchased years ago which gives us the most trouble as a result of corrosion. New meters, for the most part, are designed better and the metals used are more likely to be similar.

Replacement parts for old meters are watched very carefully. Where yellow brass cannot be used we specify composition of metals wanted.

In conclusion, I want to say just a word in regard to ordering meter parts. Up until several years ago we encountered a great deal of trouble by getting parts which we did not want, which resulted in a general delay. To correct this trouble a special meter parts order form was printed. This form had a place for size, make, model, type, serial number, meter part catalog number, etc. An individual order blank is used for parts desired for each size, make, etc. We found this leaves little room for the meter manufacturer to have any doubt as to what part is wanted. The receipt of meter parts was also more easily accounted for.

As you have already observed, I have made no attempt to describe all details. Some of the points mentioned, if properly discussed, would require a report the length of the entire paper.

My theory on water meters is: *Buy the best meter you can—and when it comes to the repair shop turn it out as good as, or better than, a new meter.*

(*Presented before the Southwest Section meeting, October 14, 1935.*)

FACTORS IN MAKING RATES

By Marvin C. Nichols

(Consulting Engineer, Fort Worth, Tex.)

Public utilities, as they effect life in our metropolitan areas, may be classified as: (1) Telephones, (2) electric, light and power, (3) gas, (4) water, and (5) to a more limited extent, sanitary sewers.

The development of public utilities during the last quarter century has been most interesting. Today the telephones, electric light and power systems, and gas systems are usually parts of an integrated intra-state or inter-state system. With comparatively few exceptions, these public utility systems are privately owned.

Rate making for these public utilities is complex, due partly to the transmission lines connecting the various town plants. The factors to be considered in rate making for these utilities have been developed largely by a process involving court decisions and regulatory rulings.

On the other hand, water systems and sewerage systems are, in general, municipally owned. It is estimated that 70 percent of the water systems in Texas are municipally owned, with 30 percent privately owned. The percent of municipally owned sewerage systems is even higher. Usually no inter-city connections exist, thereby simplifying the problem.

In considering the factors involved in making rates, a further distinction could be made between privately and municipally owned plants. For the purpose of this discussion, however, the factors involved in making rates for both municipally and privately owned utilities are discussed. Some factors involved in making rates for privately owned utilities are not involved in the making of rates for municipally owned utilities. On the other hand, some additional factors are involved in the making of rates for municipally owned public utilities which do not appear in the making of rates for privately owned utilities.

Texas has never adequately regulated the rates to be charged by

public utilities. Adequate regulation, which includes rate making, has to some degree, at least been opposed by companies, municipalities and the consumers themselves.

In Texas, the rates to be charged by the utilities in incorporated towns are determined originally by the city administrative body. This applies to telephones, electric light and power, gas, water, and sewerage systems.

The State has provided, through the Railroad Commission of Texas, a review of the rates to be charged for gas distribution in all cities. That is, the rate for gas is originally determined by the municipality and the Railroad Commission is a Board of Review of this rate.

The State has not provided any Board of Review for any other public utility. The recourse of either the company or the city is to the State and the Federal Courts.

The Railroad Commission does not have jurisdiction over electric transmission lines, telephone or other wire communications. In Texas, the Railroad Commission has original jurisdiction over natural gas pipe lines engaged in intra-state commerce.

In effect, Texas has as many regulatory bodies for public utility matters as we have city commissions or city councils. These councils and city commissions change with remarkable regularity. As a result, regulation or rate making in Texas has not been developed to the extent that it has been in other states.

The factors to be considered in making rates are as follows:

1. Fair value of the property
2. Operating and maintenance charges
3. Free services
4. Financial structure
5. Fixed charges
6. Depreciation, replacements, amortization, and obsolescence
7. Rate of return
8. Consumer use
9. Ability of customer to pay
10. Political considerations
11. Method of financing extensions
12. Contributions from ad valorem taxes
13. Fire hydrant rentals
14. Competitive situations
15. Accounting system
16. Rate structure

FAIR VALUE OF PROPERTY

One of the most important considerations in the determination of the rate to be charged by a public utility, is the determination of the fair value of the property. Many years ago, in the case of Smyth vs. Ames, the Supreme Court of the United States laid down the principle that a public utility was entitled to earn a fair rate of return upon a fair value of the property. In that case, however, the Supreme Court did not define what it meant by "fair value."

During the subsequent years, *fair value* has from time to time been approached from the standpoint of:

1. Prudent investment
2. Historical cost
3. Book cost
4. Value based on earnings
5. Spot reproduction cost

Generally, in the determination of fair value, some weight is given to all of the elements above enumerated. As of the present time, however, it is generally considered that the leading factor in the determination of fair value is the reproduction cost new, using spot prices as of the date of the inquiry and deducting the accrued depreciation.

Much could be said as to the fairness of using prudent investment, historical cost, book cost and reproduction cost as the basis for determining fair value. It is generally conceded that the use of a method involving capitalization of earnings would not be proper when applied to a utility enjoying a virtual monopoly and virtually without competition.

Historical cost in general would differ from prudent investment to the extent that the historical cost would include both prudent and imprudent investments.

Book cost might not be either prudent or even historical cost. Book cost would simply be the cost of the property as reflected on the investment ledgers of the company. In some instances, book cost includes a write-up of appreciation based on a valuation of the property at some previous time. Book cost, in some instances, represents the amount paid by the present owners to an original owner and therefore would not represent the historical or prudent investment cost. Usually book cost is not a reliable measure of fair value.

As stated before, reproduction cost new at spot prices prevailing, less depreciation, is the factor generally given most weight in the determination of fair value. This method (spot reproduction) was given approval by the U. S. Supreme Court in the Indianapolis Water Company Case. Reproduction new less depreciation was approved by the Supreme Court in the Knoxville Water Case.

In the determination of reproduction cost, it is necessary that an inventory of the property be made. It is usually found that very few public utility systems have adequate inventories of their properties. Usually a map can be found of the properties but only on infrequent occasions can an inventory be found. Utilities whether municipally or privately owned should prepare adequate inventories of the property.

Without endeavoring to elaborate upon the items generally included in a reproduction cost new valuation of the property, let it be sufficient to enumerate them as follows:

1. Valuation of the physical property reflected in the inventory and used and useful in the public service.
2. Preliminary and organization expense.
3. Administrative and legal expense during construction.
4. Engineering and supervision expense during construction.
5. Interest during construction.
6. Working capital.
7. Going concern value.

A proper determination of the above factors will give the fair value of the property as represented by reproduction cost new.

Engineers who have had little experience in rate matters, are sometimes prone to overlook the items referred to as collateral construction costs and non-physical cost, enumerated above 2 to 7 inclusive.

OPERATING AND MAINTENANCE CHARGES

The following is an excerpt from R. C. S. of Texas, 1925, Art. 1109a, page 330:

"2. Whenever the income of any water system shall be encumbered under this Act, the expense of operating and maintenance, including all salaries, labor, materials, interest, repairs, and extensions, necessary to render efficient service, and every proper item of expense shall always be a first lien and charge against such income. The rates charged for services furnished by any of said systems shall be equal and uniform, and no free service shall be allowed except

for city schools, or buildings and institutions operated by such city, and there shall be charged and collected for such services a sufficient rate to pay for all operating, maintenance, depreciation, replacement, betterment and interest charges, and for interest and sinking fund sufficient to pay any bonds or notes issued to purchase, construct or improve any such systems or of any outstanding indebtedness against same. No part of the income of any such system shall ever be used to pay any other debt, expense or obligations of such city, until the indebtedness so secured shall have been finally paid."

Obviously, one of the most important factors to be considered in rate making, is the item of operating and maintenance charges. Where the system is publicly owned, the manager seldom has a comparative guide on the operations of other similar plants. If data are available on similar plants he can test the reasonableness of his operating and maintenance charges. A careful analysis of all expenditures should be made by the manager to the end that the operating and maintenance charges are kept to a minimum consistent with good operating practice. It has been observed during the recent depression years that managers have been giving more attention to a reduction in their operating costs. Operating costs can be reduced by the installation of new pumping equipment, installation of elevated storage, installation of new mains, and other relative improvements.

It has also been observed during recent depression years, that many cities have not maintained their plants in as good a condition as is their normal practice. This may be false economy in the long run. Some operators have also decreased the use of softening chemicals. This may result in increased operating charges later.

Generally, the operating and maintenance charges amount to about one-third of the total expense, the fixed charges amounting to approximately two-thirds. Obviously, this is one of the most important factors involved in making rates.

FREE SERVICES

There should be no free service in a public utility system. If the system is privately owned, the city should pay for all the service rendered by the company to municipal departments. If the system is municipally owned, the other city departments should pay for such services as are rendered by the utility department. The extent to which free services are rendered must necessarily be reflected in the rate to be charged the ultimate consumer. The ultimate consumer

should not be required to pay a rate which would permit the furnishing of free service to some municipal department.

FINANCIAL STRUCTURE

The financial structure of a privately owned utility is not a factor to be considered in making rates. The financial structure is a factor to be considered if the system is municipally owned. Most public utility systems in Texas that are municipally owned, have in the past, been financed by the issuance of ad valorem tax bonds, although in some instances, the utility, has through its earnings, taken care of the fixed charges on the bonds. In general, the city does not know the extent to which the utility is self-supporting. At the time rates of a municipally owned utility are under review, the financial structure should be if possible improved to the end that its position may be improved.

In the determination of a proper rate to be charged for services, if the system is municipally owned, it is necessary to determine the fixed charges against the utility in question. This involves a determination of the outstanding bonds, both ad valorem and revenue. In consideration of this factor, it might be pointed out that during the last few years refunding of bonds has considerably reduced the fixed charges. This, of course, should result in a direct effect upon the rate structure.

If the utility is privately owned, the amount of fixed charges does not directly affect the rate to be charged for the service. This factor then becomes, under these circumstances, involved in the consideration of the rate of return.

In the larger cities revenue bonds bearing 3 to $4\frac{1}{2}$ percent have been sold recently, while ad volorem bonds in recent months have been sold on a basis as low as 3 percent.

DEPRECIATION, REPLACEMENT, AMORTIZATION AND OBSOLESENCE

This is a factor that is generally overlooked or neglected in the determination of rates for a municipally owned utility. If the utility, however, is operating on a business-like basis, this factor should be taken into consideration.

A depreciation reserve should be maintained. This reserve would be intended to provide funds for the replacement of worn-out equipment, obsolete equipment, drilling of new wells to replace abandoned ones, replacement of lines deteriorated through age, and replacements

due to changes required as a result of other public improvements, and changed conditions.

A well operated water plant would provide an annual fund of approximately 2 percent per annum of the rate base.

RATE OF RETURN

In a municipally owned utility this is not a factor to be considered in the making of rates. It is a major factor to be considered in the making of rates for a privately owned utility. The rate of return is generally applied on the fair value of the property and not on the financial structure of the company. The rate of return allowed should be sufficient to attract a free flow of capital to the company.

Under the recent court decisions, a rate of return of from 6 to 8 percent, based on the fair value of the property is generally considered adequate.

CONSUMER USE

In the determination of a proper rate, it is necessary to consider the consumer use of the commodity being served. That is to say, an analysis should be made of the sale of water in various brackets. The manager should know the number of customers using as an illustration less than 3,000 gallons per month; the number of customers using from 3,000 to 8,000 gallons per month; the number of customers using from 8,000 to 18,000 gallons per month, and continuing through the brackets of the rate structure. The manager, of course, should also know the average use for each customer in the respective brackets.

The customer use should also be determined as distinguished between commercial, domestic, industrial, and for water property, irrigation.

With this information in hand the manager or rate making body can determine the probable gross revenue based upon any rate for the particular bracket.

ABILITY OF CUSTOMER TO PAY

In the determination of any rate, the law of diminishing return must necessarily operate. The customer, no matter how great his need for a certain commodity or service may be, will in many instances, find himself unable to pay a rate beyond his budgetary means. The arrangement of the actual rate itself should take into

account the respective abilities of customer brackets to pay. This should not be construed to mean that the rate should be based on the ability of any group of customers to pay, but it does mean that practical consideration should be given to the type and class of customers in any given city.

POLITICAL CONDITIONS

Obviously, political considerations are not a proper factor to be considered in making rates. As a matter of fact, however, most rates have been established by giving undue weight to this factor. A statement frequently made by governing city administrative bodies is to the effect that the *small consumer* should get the benefit of *any reduction in rates.* This position is not always sound from a rate making standpoint but is always sound from a vote-getting standpoint. This determination on the part of city commissioners and city councilmen to provide a low rate for the small consumer has operated to eliminate, in many instances, a service charge and has also reduced in many instances the minimum bill requirements. Within practical limitations, each customer should pay his pro-rata share.

This political factor has also operated in certain instances to secure for large consumers, a disproportionate reduction in rates. This is particularly true under many so-called business administrations where large interests are influential in administrative councils.

It is to be hoped that in the not too distant future political considerations may be banned from the procedure of making rates.

METHOD OF FINANCING EXTENSIONS

In the determination of a proper rate, if the system is municipally owned, it is necessary to consider the method of financing extensions. If moderate extensions are to be financed from current revenues, then the rate must be sufficiently high to provide funds for these extensions. While this method of financing extensions simplifies the financial problems of the growing municipally owned utility, it is of doubtful soundness in making rates, as under this financing method the present customers pay for the capital investment to serve new customers.

Under privately owned utilities, extensions would be financed by new capital and under proper regulation, the rate would not be sufficient to provide funds from operating revenues with which to make extensions and betterments.

CONTRIBUTIONS FROM AD VALOREM TAXES

In a municipally owned utility, particularly water and sewerage systems, some contribution from ad valorem taxes can properly be made. This is based on the theory that the construction of the utility has enhanced the value of the adjoining real estate.

From the standpoint of water systems, it is believed that some 30 to 50 percent of the fixed charges of the system could properly be charged to fire protection and paid for from ad valorem taxes.

This division is further borne out by the fact that the Public Works Administration has financed self-liquidating water systems in Texas on a 70–30 basis and on a 55–45 basis. This has permitted the financing of water systems that could not be financed 100 percent on a self-liquidating basis. In other words, the grant made by the Public Works Administration has taken the place of the usual ad valorem tax bonds.

FIRE HYDRANT RENTALS

It is usual to make a charge for water furnished the City through the Fire Department. This is usually in the form of an annual hydrant rental varying from $25.00 to $100.00 per annum. Competent engineers and water works superintendents estimate that from 10 to 20 percent of the gross revenue should be obtained from this source. It is a factor to be considered in making rates.

COMPETITIVE SITUATIONS

In waterworks systems industries frequently develop their own water supplied, and yet may desire water for stand-by service.

In gas systems a customer may desire to burn oil when oil is 50 cents per barrel and will change to gas when oil is $1.00 per barrel.

Rates must be arranged to meet these and similar competitive situations. These factors must be considered in making rates.

ACCOUNTING SYSTEM

The above discussion is predicated largely upon an accounting system adequate for the needs of the particular plant. The accounting system need not be elaborate, but should be in sufficient detail to permit an intelligent presentation of the factors necessary to be considered in the determination of a proper rate structure.

RATE STRUCTURE

The rate structure determines the amount to be paid by individual customers. The form of rate structure to be set up depends upon all the factors involved in the determination of rates. It is generally considered good practice to establish a rate structure having a minimum bill which permits a limited use of the commodity being sold. This minimum bill is based primarily upon the theory that each customer should pay a service or demand charge.

It is also considered good practice to arrange the rate structure in such a way that with increased use of the commodity or service, the rate per unit of commodity or service declines.

Based on water service, a rate structure of the following typical form would be suggested:

Minimum bill (which includes up to 3,000 gal.)	$1.50
Next 5,000 gallons	0.30
Next 10,000 gallons	0.25
Next 20,000 gallons	0.20
All over 38,000 gallons	0.18

The above is based on monthly bills.

In conclusion I would stress the following points:

1. Determine what property you have and what is its fair value.
2. Re-examine the adequacy of your accounting system.
3. Make your rates sufficiently high to permit the conduct and operation of the utility on a sound financial basis.
4. Carefully study all factors before promulgating a rate schedule.

(*Presented before the Southwest Section meeting, October 17, 1935.*)

UTILITY ACCOUNTING METHODS

By Raymond E. Lee
(Assistant Secretary and Treasurer, Houston Lighting and Power Company, Houston, Texas)

Before the accounting machinery of a utility can operate, it is necessary for the public to agree to subscribe for the service which it is set up to render. Therefore, the customer is very courteously asked to sign an application for service, and while the clerk is filling in the necessary information on the application form, her questions and the applicant's answers are picked up by a microphone on the counter, transmitted over wires, and reproduced over loud speakers to the Credit Department, where a Credit History Card Visible File is maintained, and to the Contract Bureau, where a Street Card Visible File is maintained showing the status of all services. While the application Clerk is completing the application, the Credit Clerk is checking the credit files for the possibility of applicant having established a poor credit record with the Company during a prior period of service. A number of things could reflect upon the applicant's credit rating, such as delinquency in payment, necessity of sending collector, necessity of discontinuing service on account of non-payment, the use of diverted current, and the issuance of checks dishonored at the bank.

Should the Credit Clerk find that the applicant has established a good credit rating with the Company during a prior period of service, a green signal is flashed on the dictograph set at the application counter, and the Clerk accepts the application without deposit; but, should the Credit Clerk flash a red signal, then the applicant is asked to put up the required amount of deposit, for which the Company issues its regular form of receipt or deposit certificate. The deposit may be refunded after satisfactory rating has been established by the customer. The credit rating is considered satisfactory by the Company, if the customer has paid his monthly bills within the discount period for 24 consecutive months. If the customer applies for a refund of his deposit and presents his original certificate, his record is immediately checked and if found satisfactory, he is asked

to endorse the deposit certificate, which endorsement is verified with original signature on Credit History Record Card or Application for Service. Should the customer lose his original certificate, the Company has a regular affidavit form which the customer is asked to sign when application for refund is made. The Deposit Register is then checked to make certain that the deposit has not been refunded previously. If the record shows that deposit has not been previously refunded, the clerk immediately makes pencil notation thereon to the effect that refund is being made and refund date. This precautionary measure insures the Company against making duplicate refund of deposit pending the time of posting deposit refunds to the register in ink each day from Accounts Payable Voucher reimbursing the cashier for such disbursements. Should the applicant have an *unpaid balance* at a former address, the Credit Clerk signals the application clerk to take down the receiver and listen for instructions as to how to proceed further. The instructions, usually, are to inform the applicant that the Credit Department wishes to discuss the application with him, and he then proceeds to the second floor, where a representative of the Credit Department meets him and handles the matter to a conclusion. In the interim, the application form is forwarded by vacuum tube system from the Application Counter to the Credit Department.

The Credit Department is able to locate a name in the file and give a definite answer in six to ten seconds time. This is done through the use of the Soundex System of indexing names in visible files.

The information sent to the Credit Department is also sent to the Contract Bureau, which, in turn, checks the service location to determine if the service for the last customer at that address has been disconnected, and if so, signals the green light to the Application Clerk. If the service for the last customer at that address has not been discontinued, the information is telephoned to the Application Clerk, and if applicant informs the clerk that the premises are vacant, the clerk issues a discontinuance order for the customer previously at that address. The main reason for checking service with the Contract Bureau is to be able to answer the applicant's question as to when he may expect the service to be turned on. In all cases where there is no construction work to be performed, and no delay likely to be encountered due to City Permit not having been issued, and in case service has been applied for prior to 3:00 P.M. on week days, or 11:00 A.M. on Saturdays, the customer is assured his service will be con-

nected that day. Should application be made after the aforementioned hours, the customer is informed that service will be furnished early the next morning.

After the applications have been accepted, they are transmitted to the Contract Bureau, where the Street Card file is immediately posted, showing, for each particular location, the application number and the name of the new customer. The next step is to arrange for the prompt connection of service, and the Contract Bureau transmits the applications to the Service Department for execution. However, in view of the fact that the Contract Bureau maintains several investigators in the field, service connections not requiring the attention of a lineman or an electrician are made by them. The investigators report to the Contract Bureau by telephone at stated intervals during each day, and they are instructed to make such connections in the territory in which they are working. In this way, we are enabled to give many of our customers immediate service.

The applications sent to the Service Department are executed as promptly as possible, and much time is saved by transmitting the necessary information by telephone to the Service men who are located in strategic points in the city, and who call for orders at stated intervals during each day.

When the connections are made, the necessary information, such as meter number, constant, reading, date connected and by whom, is entered in the installation section of the application form, which is then returned to the Contract Bureau, where the posting to the street card file is completed. The applications are then transmitted to the Customer's Accounting Department in order that the proper changes may be made on the meter index sheets, and on the stencil plates, from which the register sheets and bills are printed thereafter. As each order is entered in the records by each Department, it is so noted by showing date entered, and by whom, in the respective blank spaces provided for this information.

Thus our accounting records are started on their way.

METER INDEX SHEET

The first essential accounting record is the meter index sheet, which carries the necessary information showing the customer's name, address, company meter number, meter constant, location of meter and date of installation. The sheet also provides space for the classification of service and the rate schedule on which the account is

to be billed. Since the meter index sheets are arranged in the meter book in geographical sequence, the address remains fixed; provision is made, however, for as many as four successive customer's names on each sheet. There is also provided on the meter index sheet space for the coding of the amount of customer's deposit, on which 6 per cent per annum interest must be paid to the customer in the month of January of each year, in accordance with the State law. By means of this coding being shown on the meter sheet, the billing operators, when billing services for the month of January, may readily calculate the amount of interest due each customer, and enter same as a credit on the bill. The meter index sheet provides for two years of meter readings, starting at the bottom of the sheet and building up so that readings will be in their natural position to facilitate the subtraction of the previous reading from the present reading.

The meter index sheets are arranged in geographical order by streets and are grouped in routes consisting of from 250 to 350 sheets. Each route is a day's work for a meter reader.

METER READING SCHEDULE

Our Company employes seventeen men who read meters twenty one days in each month, according to a schedule prepared monthly in advance. Routes of meters must be read on the day shown on the schedule, as the billing, delivery of bills, discounts, collections and delinquent notice schedules are all regularly timed and based on the date the meters are scheduled to be read.

The meter reading schedule purposely provides for the routes to be alternated among the readers each month; that is, it is not possible for the same meter reader to read the same route for a period of seventeen months. This system has many advantages. In alternating the reading of routes, the readers are practically covering new territory each month, which the company believes keeps them on the alert and makes them more careful and accurate in their work. We are glad to say that out of the 115,000 meters read every month, very few errors are made.

In the regular order of routine, after the meters are read each day, the routes are carefully scrutinized the same day by checking clerks. Orders are then issued for the necessary investigations and pick-up reading on meters, which the meter readers were unable to secure. Such orders are executed and worked into the meter books the second day. On the third day, the meter books are sent to the Billing

Department, where the bills and ledger record are prepared by mechanical means on Billing Machines. There are five billing machine operators in the Houston office, and one in the Galveston office, who work five days a week. All operators work on a wage incentive plan. A certain base is paid each operator for a minimum monthly bill production of 14,000 bills. For all bills produced over the 14,000 per month, the operator receives a bonus of one-half cent per bill. This encourages the operator to speed up her production, but, since speed is not of much value unless accompanied by accuracy and neatness, the operators are penalized five cents for each mistake made.

I have tried to refrain from using statistics and figures in preparing this paper; however, I believe it will be very interesting to know that our billing machine operators show an average production of 18,345 bills per operator each month, averaging 140 bills per hour of actual working time for each operator, with an average of 68 errors per operator per month.

CUSTOMERS LEDGER RECORD

Our system of accounting provides that the register sheet (Customers Ledger) be a by-product of the billing; therefore, the register sheets are prepared simultaneously with the billing.

The bills are addressed on an addressing machine and the register sheets are also run through the addressing machine, using the same stencils to print the customers' names on the lines provided for that purpose. The stencils are arranged in the same geographical order as the meter index sheets. Therefore, when the billing clerk starts billing, the customers' names follow through in the same order on the meter sheets, the bills and the register sheets. The billing machine operator places a non-smear carbon sheet on the face of a register sheet, adjusts it in the machine, types at the top of the sheet the date the meters are read, date billed, district number, page number and her name. Then the sheet is turned up to the first customer's name, which is visible, and sight check is made to see that the names on the register sheet, meter index sheet, and bill agree. The bill is then dropped in place and the billing operation is begun. You may or may not be familiar with lighting and power and gas billing, but it does not differ materially from the billing of water accounts, so it will be worth our while to cover this phase step by step in detail.

First, after the bill is placed in the machine, the connected load code

is shown, then the class of service code, the present meter reading, and the previous reading. These readings register in vertical adders on the machine, and automatically and simultaneously register in a cross-footing register. The present reading automatically adds, and the previous reading automatically subtracts, leaving the net consumption in the register. The meter reader's subtraction of energy used is then picked up from the meter index sheet by the operator, and entered in the next column, registering in a vertical adder, at the same time automatically registering as a subtraction in the cross-footer, clearing the register for the next operation.

At this point, the operator depresses the proof key, and if she has correctly picked up the present reading, previous reading, and meter reader's subtraction of energy used, and if the meter reader has correctly calculated the subtraction, the proof key will actuate and the machine will print a star after the amount of energy used. If any error has been made, either by the operator or the meter reader, the machine will not print the proof star, but automatically locks. It is then necessary for the operator to locate the error and make the necessary correction, after which the proof star can be printed. This operation is a positive proof that the present reading, previous reading, and energy used, as shown on the bill, are correct and obviates the necessity for a long, detailed check of meter readings to prove their correctness.

The pre-calculated rate charts are valuable time savers, and in order to achieve the maximum of efficiency, calculations of consumption are only shown in even figures (2, 4, 6, 8, 10, 12, etc.), which allows us to cut our rate charts in half, thereby saving the operator's time considerably.

In order that our rate charts may be calculated only in even figures, it is also necessary to read meters in even figures only. For example, if the last dial on the meter has reached, let us say, three, the reading is shown as two. On the other hand, if the last dial has passed three, but has not reached four, it is shown as reading four.

The next step is to refer to the rate chart. Opposite the kilowatt hour consumption is found the total gross and net amounts, both of which are entered on the bill, automatically registering in vertical adders and cross-footer adders. Should the customer have any arrears on his account, this is the next amount entered on the bill, also registering in a vertical adder and the gross and net cross-footer adders. The next column on the bill is for any miscellaneous charges

or credits which, when entered, register in a vertical adder and also gross and net cross-footer adders. The next column is for merchandise charges and any amounts written therein register in a vertical adder and also in gross and net cross-footer adders.

The foregoing comprise the possible entries on the customer's section of the bill, and the next two columns to the right are the coupon section, in which the total gross and net amounts are picked up from the cross-footer adders, and entered. As they are typed on the coupon, they automatically clear out of the adders. The operator then depresses the proof key, and if the amounts have been correctly placed on the coupon, the key will actuate; but should the operator, through some means, have picked up the wrong figures from the cross-footers, the machine automatically locks, indicating an error has been made. It is then necessary for the operator to locate the error and make the necessary correction, after which the proof key is depressed and the star printed.

At this stage of the billing operation, the bills are complete, but are held until the proof of billing has been accomplished, after which they are released to be delivered.

The totals for all columns on the register sheet are copied from the individual adders at the completion of each sheet and typed on the last line of reverse side of sheet. The proof of billing and revenue analysis is made from the register sheet by calculating machine operators with the use of a peg board. The revenue analysis is made by taking from the register sheet the total kilowatt hours for all customers on the same rate schedule, and extending it into money according to various steps in rate. There may be several rate schedules to be applied to the various classes of service on a sheet, and as each class of service is calculated by the calculating machine operator, it is entered on a yellow analysis strip. There are three forms of analysis strips used in proving the billing and analyzing of revenue. These forms are identical, with the exception of their color. The yellow analysis strip is used to classify all revenue on one individual register sheet. There are from five to seven register sheets to the route. The pink analysis strip is used to summarize the information shown on the yellow strip, and represents one route out of the total number of 362. This total number of routes is broken down into twenty-one districts, consisting of seventeen routes each. The blue analysis strip is used to summarize the pink strips by districts. To speed up and facilitate the operation of summarizing

the information on the analysis strips, in each instance, we use the peg board. The number of bills in each rate classification is also compiled by the calculating machine operators, and entered in the space provided on the revenue analysis strips.

At the end of the month's billing period, the twenty-one blue analysis strips are placed on the peg board and summarized on one master blue strip, which constitutes the total revenue for the month and the total number of bills classified by rate schedules.

This operation not only furnishes a complete breakdown of revenue by classes and rate schedules, but also proves the billing operation and insures against the possibility of releasing any bills incorrectly calculated. After this operation, the bills are then released to the Treasury Department for delivery to the customers, a period of five days after the reading of the meter.

We shall now go back and take up the discussion of the register sheet after the proof of billing and revenue analysis have been accomplished. The register sheet is now ready to be inserted in the current binder for ready reference in posting cash, preparing duplicate bills, writing delinquent notices, posting final billings and for such other purposes as the customers' ledger might be used.

Following across the register sheet, the remaining columns will be described in their order.

First, the forfeited discount column is used for posting the forfeited discounts on the thirteenth day after date of bill for such accounts as remain unpaid.

The next three columns are provided for posting of final bills to customers' accounts whose service has been discontinued. These columns show the date of final bill, the kilowatt hour consumption, and the amount. Final bills generally cover a fractional part of a regular billing period, and the amount of such bills must be calculated by the Final Bill Clerk, and in order to facilitate the revenue analysis, the kilowatt hours by rate steps are entered in the name section of the sheet.

The next two columns provide for the credit entries to the customer's account showing the date paid and the amount. For all accounts that are paid in full, a rubber stamp dater is used in posting the credit. In cases where partial payments are made by customers, the cash posting clerk makes the credit entry with pen and ink, showing the date paid and the amount. Any other credit posting, such as rebates or allowances, and transfers of amounts from one address

to another, are all made with pen and ink, showing reference number in the date column, and amount of credit in the amount column.

The next column provides for unpaid balances. The unpaid balances are extended on the period balancing date, which is the day following the date meter is read for the new month.

As stated in the foregoing, the forfeited discount is set up on the register sheet thirteen days after the date of the bill, and the next day, the delinquent notice is prepared in triplicate on an Accounting Machine, using continuous Fanfold Forms supplied with one-time carbon paper, for all accounts showing a delinquent amount of $1.75 or over. The original of the notice is printed in red and is mailed to the customer; the duplicate is printed in purple and is retained in the Collection Department office; and the triplicate is forwarded to the Credit History Record Department for entry on the customer's Credit History Record Card.

Our whole system is designed to operate on the unit plan basis; therefore, we have our accounts broken into units, and each unit clerk is responsible for the unit controls under her supervision. It is her duty to make all debit and credit transfer entries affecting the accounts in her unit controls; to see that all final bills are prepared on schedule date and posted; to set up the forfeited discounts on the proper schedule date; to extend the unpaid balances on accounts on the period balance date, and to see that all accounts under her supervision are in balance with the general controls.

HANDLING BILLS

We shall now take up the handling of the bills and the receiving of payment therefor.

The completed bills, after their correctness has been proven by the analysis clerks, are sent to the mailing room, where, by mechanical means, they are folded, placed in envelopes and sorted for mailing or delivery by hand, as the case may be. The ones to be mailed are readily detected by proper notation placed on the stencil plate, which, in turn, is printed on the bill. We use the open window envelope, which is the most economical of its kind.

The bills to be delivered by hand are turned over to the Collection Department for delivery to the customers. Five bill deliverers start out on their mission the first thing in the morning. The routing of these bill deliverers is carried on in the same manner as the routing of the meter readers; that is, they do not deliver the same routes every

month, but, on the contrary, are given different routes to deliver each month. Each bill deliverer handles approximately eight hundred bills each day.

May I digress for just a moment at this point? No doubt, you are all aware of the postal regulations regarding the depositing of bills or circulars in mail boxes and are probably wondering how we satisfactorily meet all requirements of the Post Office Department.

Our bill deliverers, all of whom are full time salaried employes, are carefully instructed not to place the envelopes containing the bill in the customer's mail receptacle, but, wherever possible, to slip the envelope underneath the door; if that is not possible, to place it between the screen door and the house door, and should the screen door be locked, to find some means of securely placing the envelope somewhere around the screen door so that it will not be blown away. We are very well pleased with this method of delivering our bills, as the comparatively few requests for duplicate bills at the office indicate to us that practically all bills are received by our customers. You will be interested to know that the statistics for the Houston Division show that, over the past twelve months, we have had the small average of 4,298 duplicate bills issued per month; whereas, we have mailed or delivered an average of 81,000 bills per month. This is 5.3 percent.

In accordance with the terms of the ordinance governing the application of electric lighting and power rates in the City of Houston, the Company's customers have ten days in which to pay their current month's bill on a net basis, but if the net amount is not paid within the ten day period, the gross rate applies. If the account is still unpaid four days after the net payment date expires, the delinquent notices are prepared and mailed. The customer is then given five days in which to pay the account. The delinquent notice calls the customer's attention to the fact that if the account is not paid within five days, the service will be subject to discontinuance without further notice. In order to give the customer the benefit of the doubt, and to allow two days for receipt of any mail remittances, the collector, or cut-out man, does not call on the customer until seven days after the notice has been mailed. If the collector is not successful in making the collection, the service is discontinued before he leaves the premises. I should like to impress upon you, however, that should any customer have illness in the family, or some other urgent reason why the service should not be cut off, the collector immediately gets in

touch with the office for further instructions, which, in practically all cases, are to the effect that the service shall remain temporarily connected, pending a definite arrangement.

Next in order comes the receiving and accounting for checks and cash in payment of accounts receivable. We have four cashiers on duty regularly, which number is augmented by another cashier on rush days such as naturally follow the first and sixteenth of the month. For the convenience of customers who find it necessary to request duplicate or final bills, one of the cashiers is maintained on the second floor, where such bills are issued.

Our cashiers, and all other employes who contact the public, either in person or by telephone, are carefully selected and coached in the art of being courteous and polite to our patrons.

Each cashier has a separate cage similar to that of a bank teller, from which she may serve the public. We also have one mail remittance cashier who handles all checks received in the mails. It is his duty to see whether each check received is accompanied by a coupon, and, if not, he checks the signature on the check with the Credit History Record to determine the geographical location of the account. After this has been determined, he looks up the register sheet on which the account is carried, and prepares the coupon to be passed through the day's receipts. After all checks have been identified and coupons attached, the coupons are run through the Cash Register for recording in the day's receipts. All cashiers, including the mail remittance cashier, have a Cash Register Machine. Each bill and stub handled by the cashiers is placed in the Cash Register and while the customer is tendering cash or check in payment thereof, the cashier depresses the proper keys to register the amount and district number. Upon receipt of sufficient cash, or check in correct amount, the actuating key on the register is depressed and an indelible receipt impress is placed by the cash register on both the bill, which is to be returned to the customer, and upon the coupon, which is automatically cut off and dropped into a locked box. At specified periods during the day, a representative of the Accounting Department unlocks the coupon compartments of the cash registers and removes the coupons deposited therein, at the same time noting from the register's visible audit sheet the number of the last transaction. This is for the purpose of satisfying the head cashier that the coupon clerk has removed and actually received all coupons up to and including that number.

At the time the bills are receipted by the Cash Register, the amounts are automatically accumulated in the register by district controls.

After the coupons are removed from the cash register by the representative from the Accounting Department, they are first sorted as to district controls, after which they are placed in order as to account number. For the purpose of balancing the coupons with the audit sheet removed from the Cash Register, the cash posting clerk runs a tape on all coupons in his possession, by district control. He then posts to the credit of the customers' accounts, on the register sheet, the amounts shown on the coupons as having been paid, and the total of the cash posted is balanced with the amount recorded on the cash register audit sheet.

The cash received during the day is posted by clerks who report for duty at 6:00 P.M. daily and remain on duty until their day's work has been completed and balanced with the controls. This method of posting cash, eliminates the possibility of mailing delinquent notices or sending the collectors out on accounts which have been paid.

DISCONNECTION OF SERVICE

The routine for disconnection of service is similar to that followed in the connection of service, which I have previously outlined. When the disconnection order shows that the premises will be immediately re-occupied, the meter is read and the service remains connected for the succeeding customer. Disconnections are made as promptly as possible, in order that customers moving away may receive a final bill and the refund of deposit, without delay. When a customer moves from one address to another address, the final bill, and current bill if unpaid, are transferred to the new address.

When the disconnect orders are received in the Customers Accounting Department, the proper changes are made on the meter index sheets, and the customer's name, and mailing address, if any, are blocked out on the stencil plates. Such stencils will then show the service address, and district and account numbers, and they are placed in their proper positions in the stencil file. When the Customers Account Register sheets are printed, each location connected to the Company's lines, whether in service or not, will appear in its proper position, based on account number. In this way, a complete record of the status of all services is constantly visible.

In handling the customer's accounts on the register plan, the

procedure provides for the daily compilation of rate statistics, which, when accumulated for a month, forms the nucleus of the information required from the Customer's Accounting Department for the General Financial Report.

A system for handling the accounts of public utility customers must embrace many features which do not enter into the field of general accounting. Such a system must be built around the thought of giving the customer the best and quickest service at all times, and the detailed records must be kept in such a manner that all necessary information may be accurate and instantly available.

It is apparent that it is not possible to go into the minute details of a system devised for public utility customers accounting, in the time allotted for the presentation of this paper. I have merely endeavored to outline the major points, in order to give you a picture of the structure of the system, and I should like, at this time, to extend an invitation to you to visit the office of the Houston Lighting & Power Company, if you should care to go further into the details of the system I have outlined. We shall be happy to explain any points which may not be clear, and any items which may be of special interest to you.

(Presented before the Southwest Section meeting, October 16, 1935.)

DISTRIBUTION SYSTEM MAINTENANCE

By Henry Magnus

(Secretary and Superintendent, Board of Municipal Works, Winona, Minn.)

The distribution gridiron is a very important part of the water supply system of every city. It is the transmission line which carries water for fire protection, industrial uses and domestic needs and should be kept in a safe, clean, sanitary condition at all times.

The system should be carefully planned, providing for well proportioned trunk and feeder mains, not only for present needs but future growth as well, so that adequate supply and pressure may be constantly maintained, satisfaction may be assured to consumers and a low fire insurance rate may be provided for the City.

The mains should preferably be cast iron pipe (we use Class "B" Bell and Spigot on account of its long life and dependability) laid with sufficient cover (we cover to a depth of 6.5 feet) to guard against freezing in the coldest weather. The joints should be closely fitted so that the spigot end seats against the shoulder of the bell to prevent the pipe from moving. Pipe should be laid as straight and level as possible to keep friction at a minimum. Sufficient calking material should be used to insure a durable, tight joint.

We do not install mains smaller than 6-inch diameter in order to furnish an ample supply of water, especially for fire purposes.

In making extensions to the mains from time to time it is desirable to clean the inside of the pipes thoroughly as they are installed and sprinkle about one half ounce of chloride of lime in each length of pipe as laid. This can be flushed out at the hydrant when the extension is completed. It is a safeguard against contamination.

GATE VALVES

Gate valves should be installed in the main lines at intervals of three to four blocks in the residential section, and in every block in the business district. There should also be a gate valve inserted in each block on cross mains. This creates a small area to be shut off in

case of leaks, breaks, or other causes, and is very desirable at times when the mains are flushed, which should be at least once and preferably twice a year.

All gate valves should be enclosed in a brick or concrete manhole, provided with a cast iron ring and cover to avoid cutting pavements, and large enough for making repairs easily. They should be inspected and operated at *least once each year*.

Gate valves should not be operated by anyone but a member of the field force of the Water Department. They should be of the slow opening type, easy of operation, and kept in good repair.

Gate valves should be installed in the branch lines from the mains to fire hydrants in the business and closely built up districts, so that the hydrant could be shut off in case of damage from any cause without interfering with service lines to adjacent property, or shutting off valves on force mains. We believe, this is recommended by the National Board of Fire Underwriters.

There should be kept in stock, at least one gate valve of each of the smaller sizes such as 4-, 6-, 8- and 10-inch, for emergency purposes. This applies also to water mains and hydrants, of which there should be at least one length or unit of each size.

It is desirable to keep repair parts for all gate valves and hydrants in stock, in order to expedite repairs when necessary.

HYDRANTS

Hydrants should be placed at each street intersection in the boulevard about two or three feet back from the curb line, in order to guard against damage as much as possible from being hit by automobiles or other causes.

They should be kept in good repair constantly and should be inspected regularly twice a year, spring and fall, and monthly during the severe cold weather to guard against freezing.

They should be lubricated and packed if necessary, in the fall to insure quick operation in case of fire during the winter, and those used by the fire department from November 1 to April 1, should be promptly reported to the Water Department, which should examine them immediately to see that they drain properly to prevent freezing.

Hydrants should be supplied with 6-inch elbows and 6-inch branches to mains in order to insure full fire flow.

When hydrants are broken or damaged accidentally, we take them out of service and replace with a new one from stock. The old one

may be welded and put in stock to be used again, if it has not been damaged beyond reasonable cost of repairs.

When hydrants are repeatedly damaged or broken on any particular site where traffic is not strictly confined, we insert guard posts in the ground to protect the hydrant against future damage.

Hydrants should be used only for fire protection or other city use, such as street flushing, sewer flushing, etc. In such cases they should be provided with control valves.

No one should be permitted to operate fire hydrants except members of the Water Department field force, or the Fire Department, and the latter only during the course of a fire when the whole system is at their service and control.

Fire hydrants should be conspicuously painted so they are easily discernible to the Fire Department, especially on dark nights.

LEAKS AND BREAKS

Leaks and breaks in the distribution system should be promptly repaired so as not to cause undue inconvenience to water consumers and also to prevent waste of water which costs money.

Where sewer lines are laid near to or pass under water mains, care should be exercised to fill the trench properly with compact earth, well tamped; otherwise a break may occur in the water main through unequal settlement sooner or later which may cause an expensive repair job.

Where water mains pass under railroad tracks, it is advisable to lay them inside larger pipes under the tracks proper, to guard against breakage from vibration.

When private service lines develop leaks, the owner should be immediately notified to have a licensed plumber repair it at once. Continuous waste of water should not be permitted. Each house or lot should have its own service.

Services which have been shut off indefinitely at the curb stop, should be examined occasionally for leakage as these service pipes are apt to cause trouble from corrosion.

When services are ordered off permanently, they should be shut off at the tap on the main in order to avoid future trouble.

When the management is of the opinion that there is leakage taking place in the distribution system which they are unable to account for, it might be advisable to have a thorough survey made of the whole system by some reputable company.

We had such a survey made at Winona in April, 1931, showing a loss of 264,000 gallons per day or about 17.6 per cent of the water pumped into the mains at that time of the year.

This waste at 7 cents per thousand gallons would amount to $6,745.00 annually.

The cost of the entire survey was $2,300.00 thus showing a substantial saving to the City after all waste was stopped.

An automobile truck should be provided for the foreman or other member of the field force, to be kept at his residence at night so they may immediately answer any call. They should also be supplied with a telephone at their residence.

MAPS AND RECORDS

Finally, we have found a map drawn to scale showing the plat of the City with all water mains, gate valves and hydrants, hung on a wall in our office to be very valuable. Especially in case of emergency it can be easily determined what action to take to expedite repairs. This also applies to the Fire Department.

We keep a card index system and other records of installations of mains, hydrants, valves, taps, curb stops, services and meters. These are very important and should be included in every well managed water works system.

(Presented before the Minnesota Section meeting, October 19, 1935.)

TRUNK LINE PITOMETER SURVEY

By Leonardo No Thompson

(*General Superintendent and Engineer, St. Paul, Minn.*)

It is an unfortunate but nevertheless a true fact that the supply of water decreases with the increased demands for it. Per capita consumption increases with the increase in a city's population. A large percentage of the water supplied to our cities is lost either through consumer wastage or system leaks. In many cities consumption of water is easily double the amount which can possibly be made use of, and in a very large proportion of them, the wastage is fully one-third of the entire quantity supplied. This excessive use of water not only increases the cost of pumping unnecessarily, but adds to the expense in all parts of a water works system. The effect is noticed perhaps most of all in the reduction of pressures, since the frictional head is nearly proportional to the square of the discharge. For the same reason a moderate reduction of the consumption will result in a large increase in pressures.

The problem of waste prevention is thus seen to be one of great economic importance, particularly if that city is already close to the limits of its ability to supply water. Not only does the growth of any city make the securing of additional water supplies a matter of considerable concern, but the constantly diminishing supply of all water sources makes water conservation a matter of concern to the smaller as well as to the larger communities.

Unaccounted for water is due to three causes:

1. Errors in meter registration.
2. Pump slippage.
3. Actual loss through leaks and breaks in the distribution system and services.

It is the latter item that we are particularly interested in today. Pipe leakage is almost certain to increase as a system gets older, on account of the loosening of joints through settlement, leakage of valves, hydrants, services, and undetected leaks, and much and constant work is required to keep it to an economical minimum. It

is estimated that such leakage runs from 3 to 10 gallons per capita per day.

The waterworks superintendent or engineer builds and extends his plant with an eye to the future. In the early days, unless the superintendent was a man of unusual experience and remarkable vision, it was unlikely that he could foresee the growth that was ahead of his municipality, which would require great extensions of mains, of ample size for fire protection; full capacity pumping engine and filter plants, with all of their accessories, and so on. His city was probably jogging along at an even pace and there was little indication of the expansion which was to follow.

The consequence was, in most instances, when the increase in population and extent of territory took place, the water works was scarcely in a position to cope with the situation. Its mains in the outlying district were probably 6- and even 4-inch in size, entirely inadequate alone to supply the ordinary demands of the fast growing district, not to speak of fire protection. Moreover, the department's standpipes and reservoirs would not begin to furnish properly the capacity which was urgently needed with the steady and rapid growth of population and territory to be supplied.

Expensive readjustments and replacements became necessary. Small mains had to be ripped up and larger pipes substituted in these districts, now rapidly becoming populous centres of the city. Bigger storage reservoirs and standpipes to provide pressure had to be provided at once. Larger pumping engines, of greater capacity, had to be installed. Extensive purification systems had to be adopted, to take care of the rapidly increasing pollution of the waters.

Much of this expense might have been unnecessary if the superintendent had been possessed of sufficient experience and foresight to have realized the inevitable truth that municipalities do not remain stationary, and had provided sufficient funds to finance the larger main. There must be growth or decline, with the former movement prevalent in an overwhelming majority of cases. As the density of population along a pipe line increases the per capita consumption also increases. These two factors, particularly in periods of dry seasons and unusually heavy demands, cause rapid declines in water pressures to the home to the point where complaints from the consumer reach serious proportions. Unfortunately, a distribution system is not elastic. Its carrying capacity cannot suddenly be increased to meet sudden demands, as a street railway system can put

on more rolling stock to handle a sudden rush of travel. It becomes, therefore, the problem of all water works engineers at some time in the growth of the system to plan for the reinforcement of the distribution system through the medium of larger supply and distributing mains in the gridiron. To know where such increases are necessary and what sizes are required may be a guess or a scientific study, since, unfortunately, the intensity and direction of flows are usually unknown quantities.

THE TRUNK LINE SURVEY

A trunk line survey was recently made in the City of St. Paul Water Department by the Pitometer Company of New York City for the purpose of collecting data in regard to the flow of water in the various trunk mains and principal distribution feeders of the water works system. A complete investigation of such mains was made to learn what each was doing under operating conditions.

The work covered the following items:

Division of the system into 15 sections, and a measurement of the consumption in each for a period of 24 hours, for the purpose of providing a basis for an analysis to determine whether or not sufficient avoidable leakage existed to warrant further investigation.

Twenty-four hour measurements of the flows at all critical points of all mains 16 inches in diameter and over, together with similar 24-hour measurements on all 12-inch mains or larger leading from such trunk mains, where conditions permitted accurate measurements to be made.

Loss of head tests, where practicable, on all mains 16 inches in diameter and larger, to determine the values of "C" in the Williams-Hazen formula, as an indication of the internal condition of the mains.

Analyses of the results of the above tests and recommendations for the betterment of faulty conditions.

Details of measurements and loss of head tests are presented for each section of trunk main.

Charts showing the distribution of flow throughout the periods of measurement for each gauging point were prepared.

For the trunk main survey the measurements of the flow of water were made by the use of photo-recording pitometers installed in each test section of pipe and, where feasible, on each important intersecting main. Measurements were continuous over a period of 24 hours. From the results obtained, simple computations will give the amount

of water delivered to the distribution system between the gauging points. In this way a complete analysis of the flow of water may be made for any studies desired.

To obtain loss of head data, direct reading recording pressure gauges were used, so calibrated that the chart divisions could be read with an accuracy of about 0.5 foot head of water. By averaging the records and by intelligent interpolation, results were obtained to tenths of a foot. By transposing the recording gauges and rerunning the test immediately a considerable source of error was eliminated, the mean of the data of the double tests being used in computation. The gauges were checked after each set of test by use of a dead weight tester, and observations were corrected in accordance with these calibrations to obtain the true heads of water at the test station.

TABLE 1

Summary of data obtained from pitometer survey

Section	District	Total consumption	Minimum night rate	Percent night to total	Miles of main	Underground leakage	Industrial consumption	Under registration of industrial meters	All other uses and waste
3	1	1,230,000	730,000	59	13.6	297,000	630,000		303,000
3	2	570,000	300,000	53	18.9	102,000	35,000	14,000	419,000
9	1	760,000	360,000	47	18.7	47,000	240,000		473,000
9	2	1,940,000	960,000	50	21.3	165,000	400,000		1,375,000
Total		4,500,000	2,350,000	52	72.5	611,000	1,305,000	14,000	2,570,000

Note. All figures, unless otherwise noted, are gallons per 24 hours.

Elevations were supplied by the department close to the gauging points, from which they were transferred by the engineers to the gauges.

The difficulties in making satisfactory loss of head tests in the field are great. Slight errors in pressure readings, and in transferring elevations from the reference point to the gauge, short lengths of pipe, and low velocities, are all factors which tend to affect the accuracy of the results.

The survey was made with two objects in view. First and more important, the flow was measured in each of the various pipes under ordinary operating conditions with all valves open; and second, a loss of head test which required the closing of all side valves so that a uniform velocity could be obtained throughout the entire length of the section under test.

Our method of procedure has been to work through the trunk main system from the source of supply toward the outer limits, finally reaching the 12-inch pipes which are a combination of feeder and distribution mains. The controlling factors in tests of these smaller mains were, first, whether the information to be gained would be of sufficient value to warrant setting the necessary taps for gauging points; and, second, whether the main should be designated as a feeder or as a strictly distribution main.

In several cases the results of the flow tests are marked "No flow." This does not signify that there were no flows at these points, but that the rate of flow was too small for accurate measurement.

In order to learn definitely whether certain sections of the city had indications that leakage might exist, 24-hour measurements of the flow into fifteen such sections were made by the use of Photo Recording Pitometers.

The results of the trunk line survey have been particularly valuable in the recent design of extensions to low pressure areas. The survey also indicated that several of the 15 sections had unusually high night rates and warranted further investigation. As a result, a contract was let for a waste water survey to be made in two of the sections. *The flow survey covered the following items:*

Division of each section into two districts, and a measurement of the water consumption in each for a period of twenty-four hours.

Further investigation in all districts where excessive waste was indicated for the purpose of locating all underground leaks in the mains and services.

Tests for accuracy of all meters larger than 3 inches in diameter in place, under normal conditions without removal; and tests of large consumers for the purpose of detecting illegal use through fire lines or otherwise.

The preparation of a map of the distribution system showing locations of all pitometer gauging points, district boundaries, and other features of the survey.

Each of the two sections was divided into two districts which were isolated by closing the proper gate valves along the boundaries: and the flow of water into each district was measured for a period of twenty-four hours by a Photo Recording Pitometer installed at a gauging point on a single main through which all the water for the district was supplied. Further investigations were made at night to determine the distribution of the night rate of flow.

(*Presented before the Minnesota Section meeting, October 19, 1935.*)

A COMPARATIVE STUDY OF STANDARD METHODS OF WATER ANALYSIS (1933) AND TWO PERCENT BILE BRILLIANT GREEN LACTOSE BROTH CONFIRMATION

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Water filtration and chlorination plant operators must examine the raw and final waters routinely to assure a safe water for their respective communities. It is not always possible to secure the services of a well trained bacteriologist to make the necessary tests for the presence of the colon-aerogenes group. More frequently the operator must be general manager, operator, chemist and bacteriologist. As a result, the training of the operator consists of a brief short course designed primarily to train him in the mere routine of making a bacteriological test without any attempt to give him any fundamental training in bacteriology. It has been our experience that these men can make a dependable test providing the water tested is definitely free from the colon-aerogenes group or, conversely, is definitely positive to the colon-aerogenes group. When the unusual occurs the results are decidedly unsatisfactory. These remarks are made with no intent to minimize the ability of many operators, as the writers have only praise for many of the men.

We appreciate the fact that the current standard procedure of the American Public Health Association is one that is ideal from a research point of view for the isolation of the colon-aerogenes group. From the view-point of a trained technician interested in determining only the presence or absence of the colon-aerogenes group the procedure is cumbersome and time consuming, although generally dependable. From the view-point of the plant operator the procedure is not only cumbersome and time consuming, but also subject to error.

Some of the errors that the writers have frequently observed by plant visits and by observing the technic of the operators during short courses will be briefly discussed.

SMEARING EOSIN-METHYLENE BLUE AGAR PLATES

In order to identify the colon-aerogenes group on eosin-methylene blue agar, it is necessary that definitely discrete colonies be obtained. With few exceptions, the operators use a heavy inoculum and a minimum surface area for the spread of the material. The result is a thick continuous smear of mixed organisms from the primary lactose broth tube capable of growing on the surface of the agar plate. This heterogeneous mixture of organisms may or may not resemble the classic description of the colon-aerogenes group. The resemblance would depend upon the prominence of the colon-aerogenes group in the primary lactose broth tube and the ability of these organisms to outgrow the other contending organisms on the agar plate. Unfortunately they do not always succeed in impressing upon the smeared growth their typical colonial characteristics. Even an experienced water bacteriologist would hesitate to pass an opinion on such plates, but the operator must and does.

Secondly, even though the operator smears the plate properly and obtains the desired discrete colonies, the colonies of the colon-aerogenes group do not always attain the class-book description. Many typical strains of Escherichia do not always produce small bluish-black colonies with metallic sheen. Sometimes they are spreading and flat; sometimes they are slightly mucoid; sometimes they have a light bluish color; and sometimes they have other characteristics not typical. The same is true of the Aerobacter group. If the trained bacteriologist hesitates to name a colony, what must the inadequately trained operator do? He must place on his report a plus or a minus sign.

FISHING TYPICAL OR ATYPICAL COLONIES TO LACTOSE BROTH FERMENTATION TUBES

Two or more discrete colonies should be cultured to eliminate the possibility of missing the desired organism. Generally to save time and materials the technician sweeps the plate with his platinum needle and plants into one lactose broth tube a multiplicity of strains and species. Surely some of them must be the colon-aero-

genes group. If the colon-aerogenes organisms so swept into the tube can outgrow their competitors the tube shows positive and if the competitors stifle their growth, the tube is negative. Secondly, if discrete colonies are present on the plate, the technician cultures only that one colony that appears in his judgment to bear the greatest resemblance to the colon-aerogenes group. He may be wrong.

Our point, in these citations, is to bring out the fact that for routine procedure, whether that procedure is made by a skilled technician or an improperly trained plant operator, this technic is not dependable.

It has been our desire for some years to devise a simple procedure entailing a minimum amount of time, labor, and materials that would eliminate the personal factor of interpretation and that would be equally satisfactory in the hands of the expert bacteriologist and the poorly trained operator. It is not our intent to devise a layman test, but it is our belief that all routine procedures should be made as simple as possible, keeping in mind of course, accuracy and dependability.

For some time we have used the following procedure in the senior writer's laboratory with satisfaction:

1. Planting of the sample in lactose broth fermentation tubes.
2. Transferring by platinum loop all tubes showing gas to 2 percent bile brilliant green broth.
3. Reporting all samples positive that produce gas in 2 percent bile brilliant green broth.

JOINT STUDIES

To try out this procedure on an extensive scale a committee was appointed by the Michigan Water Purification Conference consisting of the following ten bacteriologists at the designated filter plants:

C. H. Burdick	Flint
H. T. Campion	Grand Rapids
Irving Dahljem	Highland Park
L. B. Harrison	Bay City
W. D. Loreaux	Dundee
J. C. Richardson	Saginaw
Paul Stegeman	Midland
R. L. Sensabaugh	Iron Mountain
W. M. Wallace	Detroit
H. W. Ward	Wyandotte
Wm. Cary, Jr	Detroit (Department of Health)

All samples showing gas in the primary lactose broth tubes were tested in parallel using the standard methods of American Public Health Association (1933) and the 2 percent bile brilliant green broth confirmation procedure.

The results of all tubes showing gas in lactose broth fermentation tubes are shown in table 1. A total of 10,950 positive primary lactose broth tubes were examined. The classification of these tubes was as follows: In 6,659 tubes, the positive lactose broth tubes were positive on eosin-methylene blue agar, positive in lactose broth confirmation and positive in brilliant green bile broth confirmation. In 3,420 tubes, the gas in the primary lactose broth tubes was not confirmed on eosin-methylene blue agar or in bile brilliant green broth. In 276 tubes, typical or atypical colonies on eosin-methylene blue agar failed to confirm in lactose broth and no confirmation was obtained in bile brilliant green broth. In 180 tubes, no colonies were obtained on eosin-methylene blue agar, but gas was obtained in the bile brilliant green broth. In 223 tubes, typical or atypical colonies failed to confirm in lactose broth, but gas was obtained in bile brilliant green broth. In 192 tubes, typical or atypical colonies were confirmed in lactose broth, but failed to produce gas in bile brilliant green broth.

Of the 10,950 positive tubes, (table 2), agreement was obtained in 10,355 cases, or 94.5 percent. The remaining tubes, 595, were not in agreement, a percentage of 5.5. Of these 595 tubes, 403, (67.5 percent) favored the bile brilliant green broth with incorrect results on the Standard Methods (1933). In 192 cases (32.3 percent) the typical or atypical colonies confirmed on eosin-methylene blue agar and lactose broth, but failed to confirm on the bile brilliant green broth. In these cases, if a microscope is not available to check for the presence of spores, Standard Method (1933) will produce an erroneous report, and the use of bile brilliant green broth is a distinct advantage.

The above data represent all positive primary lactose tubes for the ten filtration plants for a period of seven months, November, 1934 to July, 1935. An examination of the data (table 3) shows considerable variation among the plants as to the discrepancies in the two methods of examination. For example, in Flint, Grand Rapids, Saginaw and Wyandotte, the tests agreed 99.4 to 99.9 percent. On the other hand, in Bay City, Detroit, Dundee, and Highland Park, the disagreement ranged from 6.8 at Bay City to 11.8 percent

TABLE 1

Classification of positive primary lactose broth tubes according to behavior on eosin-methylene blue agar, lactose broth confirmation and brilliant green bile broth

		NUMBER OF TUBES
Eosin methylene blue agar	+	6,659
Lactose broth confirmation	+	
Brilliant green bile broth	+	
Eosin methylene blue	−	3,420
Lactose broth confirmation	−	
Brilliant green bile broth	−	
Eosin methylene blue agar	+	276
Lactose broth confirmation	−	
Brilliant green bile broth	−	
Eosin methylene blue agar	−	180
No lactose broth confirmation		
Brilliant green bile broth	+	
Eosin methylene blue agar	+	223
Lactose broth confirmation	−	
Brilliant green bile broth	+	
Eosin methylene blue agar	+	192
Lactose broth confirmation	+	
Brilliant green bile broth	−	
Total positive tubes		10,950

TABLE 2

A comparison of Standard Methods (1933) and brilliant green bile broth confirmation procedures

	NUMBER OF TUBES	PERCENT
Tubes in agreement, both tests	10,355	94.5
Tubes not in agreement	595	5.5
Of 595 tubes not in agreement:		
Tubes favoring bile brilliant green broth	192	32.3
Tubes favoring bile brilliant green broth with incorrect results by Standard Methods (1933)	403	67.5

at Detroit. Thus the types of water apparently play an important part in the variations that occur.

The value of a given technic is no better than the results obtained during the worst periods of its use. The periods of greatest variation in the plants, that show for the entire period of study the greatest divergence between the two tests, was for the months of November, December, and January (table 4). In four cities, Bay City, Detroit,

TABLE 3

A comparison of Standard Methods (1933) and brilliant green bile broth confirmation procedures grouped by filtration plants

CITY	NUMBER OF POSITIVE TUBES	PERCENT TUBES AGREEMENT WITH STANDARD METHODS	PERCENT TUBES AGREEMENT WITH B.G.B.L.B.	TUBES IN DISAGREEMENT	
				Percent tubes favoring B.G.B.L.B. with incorrect results on Standard Methods (1933)	Percent tubes favoring B.G.B.L.B. with questionable results on Standard Methods
Flint	998	99.9	0.1	100.0	0
Grand Rapids	1,197	99.4	0.6	85.8	14.2
Saginaw	557	99.9	0.1	0	100.0
Wyandotte	683	99.9	0.1	50.0	50.0
Iron Mountain	3	100.0	0.0		
Bay City	736	93.2	6.8	96.0	4.0
Springwells (Detroit)	880	92.5	7.5	62.2	37.8
W. W. Park (Detroit)	765	88.2	11.8	50.0	50.0
Dundee	286	92.3	7.7	45.5	54.5
Highland Park	2,891	93.0	7.0	62.9	37.1
Department of Health (Detroit)	1,954	93.0	7.0	84.5	15.5
Total	10,950	94.5	5.5	67.5	32.3

Dundee, and Highland Park, 2055 positive lactose broth tubes were obtained. Of these, 227 or 11.1 percent of the tubes were in disagreement in the two tests. At the Springwells Plant in Detroit, 29 tubes or 19 percent out of 153 positive tubes tested were in disagreement. The smallest degree of disagreement among these selected plants was obtained at Bay City where 35 or 9.7 percent of the tubes varied out of 360 tubes examined. Out of the 227 tubes from all of these plants, 82.3 percent favored distinctly the bile

TABLE 4

A comparison of Standard Methods (1933) and brilliant green bile broth confirmation procedures in a selected group of filtration plants for November, December and January

SOURCE	TIME	NUMBER POSITIVE	TUBES IN AGREEMENT		TUBES IN DISAGREEMENT					
					Total		Tubes favoring B.G.B L.B. only		Tubes favoring B.G.B.L.B.	
			Number	Percent	Number	Percent	Number	Percent	Number	Percent
Highland Park	Nov.	139	118	84.9	21	15.1	17	80.9	4	19.1
	Dec.	518	456	88.0	62	12.0	56	90.3	6	9.7
	Jan.	523	486	92.9	37	7.1	26	70.2	11	29.8
	Total	1,180	1,060	89.9	120	10.2	99	82.5	21	17.5
Springwells, Detroit	Nov.									
	Dec.	88	72	81.8	16	18.2	14	87.5	2	12.5
	Jan.	65	52	80.0	13	20.0	11	84.6	2	15.4
	Total	153	124	81.0	29	19.0	25	86.2	4	13.8
Water Work Park, Detroit	Nov.	44	38	86.3	6	13.7	6	100	0	0
	Dec.	134	125	93.2	9	6.8	9	100	0	0
	Jan.	85	67	78.2	18	21.2	9	50	9	50
	Total	263	230	87.4	33	12.6	24	72.5	9	27.5
Bay City	Nov.	85	74	87.0	11	13.0	10	90.9	1	9.1
	Dec.	153	146	95.4	7	4.6	6	85.7	1	14.3
	Jan.	122	105	86.0	17	14.0	17	100.0	0	0
	Total	360	325	90.3	35	9.7	33	94.4	2	5.6
Dundee	Nov.	8	4	50.0	4	50.0	3	75.0	1	25.0
	Dec.	33	29	97.8	4	12.2	2	50.0	2	50.0
	Jan.	58	56	96.5	2	3.5	1	50.0	1	50.0
	Total	99	89	89.9	10	10.2	6	60.0	4	40.0
Total for above plants	Nov.	276	234	84.7	42	15.3	36	85.7	6	14.3
	Dec.	926	828	89.4	98	10.6	87	88.7	11	11.3
	Jan.	853	766	89.8	87	10.2	64	73.5	23	26.5
	Total	2,055	1,828	88.9	227	11.1	187	82.3	40	17.7

brilliant green procedure. The other 17.7 percent of these tubes produced gas in the lactose broth confirmation tubes, but were negative in bile brilliant green broth. As most of these organisms proved to be lactose splitting spore formers, the results of the bile brilliant green procedure were generally correct with the standard method (1933) incorrect. If microscopic examinations had been made as recommended in Standard Methods, these tubes would then have given the same results by both methods.

These studies are, of course, based on the principle that primary lactose broth grows out the colon-aerogenes group 100 percent. This assumption is not true, but if we accept the presence of the colon-aerogenes group in lactose as our sanitary standard, then we want to use the most accurate method of demonstrating this group of organisms in the lactose broth. Two percent bile brilliant green broth confirmation, as judged by this survey of 10,950 positive tubes is the more accurate procedure of the two methods tested.

ERRORS IN STANDARD METHODS (1933)

Three forms of errors were found in the Standard Methods (1933 procedure). These were as follows:

1. The failure of the eosin-methylene blue agar to grow the colon-aerogenes group. Out of 10,950 tubes tested, in 180 cases, negative results were obtained on eosin-methylene blue agar but gas production occurred in bile brilliant green broth. In every case examined, the bile brilliant green broth yielded colon-aerogenes organisms that produced typical colonies on eosin-methylene blue agar. In Bay City in November, 10 out of 38 positive tubes containing the colon-aerogenes group, produced sterile plates on eosin-methylene blue agar. In January, 16 tubes out of 68 tubes behaved similarly. So much for the colon-aerogenes group.

2. A second marked source of error was the fishing of typical and atypical colonies from eosin-methylene blue agar to lactose broth confirmation tubes. In 223 cases, negative results were obtained whereas the bile brilliant green broth gave positive results. In most cases where the bile brilliant green broth tubes were examined, the colon-aerogenes group was isolated. Many factors can be charged as the cause of failure to produce gas in the lactose broth confirmation; for example, hot needles used in fishing colonies, insufficient inoculum, fishing of the wrong colonies, failure of the colon-aerogenes group to grow on the eosin-methylene blue agar, etc.

3. In the carrying through of spore forming lactose splitters to the lactose broth confirmation test, 192 tubes were classified, but in some instances the organisms were of the colon-aerogenes group rather than spore forming lactose splitters.

The 2 percent bile brilliant green broth as a confirmatory medium unfortunately is not perfect. It must be remembered that this medium was originally planned as a primary culture medium and not for confirmatory procedure. In some instances we found that the medium was too inhibitory to the colon-aerogenes group and in other instances, it was not sufficiently inhibitory to prevent the growth of spore forming organisms, either aerobic or anaerobic strains.

CONCLUSIONS

The results of this study have been so encouraging that the committee feels that further study of the medium to improve it for confirmatory purposes is desirable. The data tend to convince the committee that the Standard Methods (1933) are no longer desirable from a routine operating control standpoint and can very satisfactorily be eliminated from the water treatment plant control procedures. The decrease in time necessary to test a sample; the elimination of the "human factor" in the test; and the more accurate results to be obtained all favor a recommendation to the Michigan Department of Health that the Standard Method (1933) procedure for testing water be eliminated and the shorter, simplified method be adopted as the required procedure in Michigan water treatment plants.

DISCUSSION

H. E. Jordan (Indianapolis, Ind.): This report made by Messrs. Mallmann and Hepler records and analyzes the results of a well planned coöperative study made by eleven water plant laboratories, directed and checked by the laboratory staff of the senior author. The work was carried on after a preliminary announcement had been made by the Standard Methods Editorial Committee that confirmation of the coli-aerogenes group by the use of liquid media would be permitted in the next edition of the text.

The results confirm the judgment of the Committee, both to the effect that liquid confirmation is satisfactory in the majority of cases, and also to the effect that there is enough difference in results ob-

tained with certain types of water to make it necessary that the laboratory worker support his choice of material used by statistical evidence.

To those who remain lukewarm regarding the liquid medium confirmation technic, this report should give added evidence that the forthcoming optional method is rational.

AGGREGATE FIRE LOSS FOR TWENTY YEARS*

The 1935 fire loss brings the total direct fire waste of the United States during the past twenty years up to $8,413,980,536, an annual average of approximately $420,000,000. The following table gives the annual fire losses since 1916.

1916	$258,377,952	1926	$561,980,751
1917	289,535,050	1927	472,933,969
1918	353,878,876	1928	464,607,102
1919	320,540,399	1929	459,445,778
1920	447,886,677	1930	501,980,624
1921	495,406,012	1931	451,643,866
1922	506,541,001	1932	406,885,959
1923	535,372,782	1933	271,453,189
1924	549,062,124	1934	262,848,122†
1925	559,418,184	1935	255,190,657 (Est.)

* Quarterly of the National Fire Protection Association, 29: 3, January, 1936, page 191.

† A final adjusted figure for 1934 is not yet available.

ABSTRACTS OF WATER WORKS LITERATURE

Frank Hannan

Key: American Journal of Public Health, **12**: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Features of Toronto's New Water Filtration Plant. Eng. Contract Record, **49**: 195–9, March 6, 1935. Extracts given from paper by William Storrie entitled, "The Toronto Water Works Extensions" presented at meeting of Engineering Institute of Canada. Purification plant consists of 12-m.g. filtered water reservoir, over which are located filters, pipe galleries, and chemical room. Twenty filters, comprising western half of ultimate plant, have been completed, together with administration building, central concourse, and chemical section. Portion of reservoir extending to east of main concourse forms foundation for additional 20 filters, which will give plant ultimate capacity of 200 m.g.d., with rate of filtration of 105 m.I.g.d. Mixing chambers and settling basins are located to north of reservoir. Filter underdrainage system consists of cast iron laterals with brass orifices; filter beds, of 18 inches of graded gravel, 2-inch layer of coarse sand, and 26 inches of sand having effective size of 0.55 millimeter and uniformity coefficient of 1.4. Filtered water reservoir is baffled to ensure required contact period for super- and dechlorination method of taste control. Provision has also been made for use of activated carbon, if found desirable. Mixing tanks are of spiral flow type, retention period being 40 minutes. Settling tanks provide detention period of 2 hours and 50 minutes. Specifications permitted invisible leakage of not more than 60 gallons per minute from concrete filtered water reservoir, which has floor and wall area of 174,000 square feet. Leakage observed during 120-hour test was 6160 gallons, somewhat less than 1 gallon per minute. Details given of waterproofing procedure employed.—*R. E. Thompson.*

Snow Surveys as an Aid to Flood Forecast and Control. J. E. Church. Eng. News-Rec., **114**: 879–81, June 20, 1935. General discussion, with particular reference to Nevada. Data on effect of Chinook winds on snow melting are almost completely lacking.—*R. E. Thompson.*

Flexibility in Design Aids Purification Plant Operation. Paul Hansen. Eng. News-Rec., **114**: 633–6, 1935. General discussion of water purification plant design, with emphasis on provision for flexibility of operation to meet changing conditions and to permit incorporation of new methods of treatment. —*R. E. Thompson (Courtesy Chem. Abt.).*

Flushing of Water Mains. C. Edgar Brown. Eng. Contract Record, **49**:

311-3, April 3, 1935. Discussion of theoretical principles involved in flushing of water mains. Velocity at which particles become suspended in water depends upon their size and weight, diameter of particles picked up varying as sixth power of velocity. Owing to greater difference between velocity at center and that at circumference in case of large-diameter pipes, mean velocity necessary for scouring and transporting sediment is greater in larger pipes than in smaller ones. Three feet per second is commonly accepted value for minimum velocity for transporting heavy sand and silt, but velocity of 6 feet per second is considered necessary for main flushing in order to effect scouring action. Flow required for mains of different diameters, computed on above basis, are given and it is pointed out that flow obtained from hydrant may be conservatively estimated at 2000 Imperial gallons per minute. In Meaford, Ontario, where supply is obtained from Georgian Bay through infiltration basin, mains are flushed twice each year.—*R. E. Thompson.*

Flushing Water Mains. C. G. R. ARMSTRONG. Eng. Contract Record, **49:** 313-4, April 3, 1935. Brief discussion and outline of experiences in main flushing.—*R. E. Thompson.*

Water Works at Saratoga Springs, N. Y. Eng. News-Rec., **114:** 897-8, June 20, 1935. Unit prices given from 3 lowest tenders on water supply improvements at Saratoga Springs, including pumping station, filter plant, and distribution reservoir.—*R. E. Thompson.*

Largest Ammonia-Chlorine Plant Installed on Lake Crib. ARTHUR E. GORMAN, A. H. GERSTEIN and PAUL H. PETERMAN. Eng. News-Rec., **114:** 638-40, 1935. Detailed description of ammonia-chlorine treatment plant being constructed on Dunne intake crib, about 2 miles from shore in Lake Michigan, for treating water entering 68th Street and Dunne intakes (connected by foot bridge), averaging 350 m.g.d., with peak periods in excess of 500 m.g.d., serving south side of Chicago and 17 suburban communities, total population of more than 1,000,000. Crib location was chosen to provide the 1.5 to 2 hours' contact found necessary for sterilization. Feature of design was provision for storage of chemicals and supplies, crib being inaccessible for periods as long as 1 month under severe winter conditions. Supplies will be transported to crib on specially designed steel scow. Chlorine will be received in ton containers and ammonium sulfate in 100-pound bags. After much experimentation, simple and efficient type of high-capacity (3500 pounds per day) chlorinator was developed consisting of evaporator tank in which ton container is immersed in warm water, temperature of water and, therefore, pressure of chlorine being controlled by diaphragm-operated valve actuated by the chlorine pressure. Flow of chlorine is regulated by variable orifice, rate of flow being indicated by a rotameter, a relatively new apparatus for measurement of gases and liquids. Ammonium sulfate will be applied through dry feed machines. Chemical rooms will be provided with suction blower which can effect complete change of air every minute in case of leakage and, in addition, a steel tank containing sufficient 20 per cent caustic soda to absorb 1 ton of chlorine will be available for immersing leaking containers. Electric power is furnished

through submarine cable and there is also similar telephone cable by means of which operators can be instructed from control station on shore where routine residual chlorine determination will be made.—*R. E. Thompson.*

The Ohio River: Its Future as a Water Supply Source. H. W. STREETER. Eng. News-Rec., **114**: 612–5, May 2, 1935. Comprehensive discussion of pollution of Ohio River and of its effect on present and future use of river for water supply. River is 960 miles long and has drainage area of some 203,000 square miles. Mean annual flow at mouth approximates 300,000 second-feet, normal seasonal range being from 50,000 to 1,000,000 second-feet, while minimum flow as low as 10,000 (1930 drought period) has been recorded and maximum, as high as 1,500,000. Extreme fluctuations of 40 feet or more in river stage occur. Population on watershed is about 18,000,000. During past few years, growing difficulties have been experienced in obtaining water supply of satisfactory quality from river, particularly in more highly polluted sections above Louisville. Canalization, although beneficial during periods of low water in promoting sedimentation and in prolonging time of natural purification, has resulted in increased scouring of sewage matters during periods of freshets, causing higher peak loads on purification plants at such times. In general, canalization has improved conditions during low-water periods at more distant points below major sources of pollution, but has moved zones of denser pollution closer upstream to those sources. Increase in pollution attending increased population on watershed during next 10 years may bring about critical situation in certain zones of river, notably in Huntington-Portsmouth stretch, where conditions have already passed limit of safety. Desirable program of river improvement from standpoint of water supply is outlined. Full restoration is unnecessary and practically impossible. Probably most reasonable criterion of limiting pollution would be maximum capacity of efficient water purification systems. Engineering data necessary for program of remedial measures are fairly complete and well established, principal obstacles being legal and administrative.—*R. E. Thompson.*

Water Works Operation a Problem Dependent on Personnel. HARRY E. JORDAN. Eng. News-Rec., **114**: 645, May 2, 1935. Combination of licensing of operators and holding of water works schools trains men for the improvement in water supply service for which every city and state should strive and assures water works personnel the degree of security and community respect to which hardworking willing employee is entitled. Progress in these particulars by states is shown in tabulation.—*R. E. Thompson.*

Automatic Control of Deep Wells in the Smaller Water Works Systems. A. G. PEIRSON. Eng. Contract Record, **49**: 539–40, June 26, 1935. From 1911 to 1932, Weston, Ontario, derived its water supply from Humber River, water being treated by sedimentation, filtration, and chlorination. In 1932, 2 deep wells were installed 3 miles from town and filter plant abandoned. New supply does not require treatment of any kind. By means of altitude gage and regulator, deep well pump automatically starts up when level in standpipe has fallen to set point and automatically shuts down when standpipe is full, booster

pump being coincidently started which builds up pressure to between 60 and 70 pounds: and then, when level in standpipe has again fallen to set point booster pump shuts down and deep well pump again starts up: and so on. Only supervision required is weekly visit to check automatic lubricating system and to record venturi meter readings and draw-down data. Pressure may be increased to from 100 to 110 pounds by cutting in second well pump by remote control.—*R. E. Thompson.*

Shrinkage of Hydraulic Fill in Miami Conservancy Dams. C. H. EIFFERT. Eng. News-Rec., **114**: 482-3, April 4, 1935. Shrinkage data on Huffman Dam are brought up to date and similar data given for Germantown Dam. Readings on Goldbeck pressure cells in Taylorsville Dam in September, 1934, were practically identical with those in 1925 (cf. Eng. News-Record, December 18, 1930, p. 955).—*R. E. Thompson.*

600-Mile Pipe Line Across a Desert. Eng. Cont. Rec., **49**: 548, July 3, 1935. Brief description of construction of 600-mile pipe line by Iraq Petroleum Company to convey crude oil from Kirkuk, Iraq, to ports of Tripoli and Haifa on Mediterranean Sea at cost of £9,250,000. Pipe was welded throughout, coated with hot enamel, wrapped with asbestos, and laid in trench excavated by ditching machines.—*R. E. Thompson.*

Acid Mine Drainage from Bituminous Coal Mines. L. V. CARPENTER and L. K. HERNDON. W. Va. Eng. Expt. Sta., Research Bull. No. **10**: 37 pp., 1933. From Chem. Abst., **28**: 4145, July 10, 1934. Average drainage from bituminous coal mines is about 25 per cent of rainfall. In northern West Virginia it is from 500 to 1000 gallons per day per acre of coal exhausted. Drainage is usually acid; but of 2 mines in same coal vein, one discharges water with 17 p.p.m. alkalinity, while drainage from other is 30,000 p.p.m. acid. There is no direct relation between sulfur content of coal and acidity of drainage. At pH 3.0, a pronounced buffering action is exhibited by these acid waters. It is not economical to neutralize drainage with chemicals because of high cost and large volume of sludge formed. More sulfates are formed in unsterilized than in sterilized samples. *Thiobacillus thioöxidans* developed at pH 3.1. Degree of sterilization of sewage organisms by the acid water is function of contact time. Increases in plate counts on neutralized samples indicate that action is inhibitive, rather than bactericidal. Acid mine water considerably reduces biochemical oxygen demand of sewage-polluted streams. Twenty-nine references.—*R. E. Thompson.*

The Influence of Oil in Soil Corrosion. W. F. ROGERS. Oil Weekly, **68**: 12-16, February 13, 1933; Met. Abstracts (in Metals and Alloys), **5**: 39; cf. C. A., **27**: 4660. From Chem. Abst., **28**: 4026, July 10, 1934. Oil in soil materially accelerates rate of soil corrosion. Soils which would normally allow pipe life of 20 to 30 years may become so changed that pipe will be penetrated in 5 or 6 years. Action varies both with type of soil and with quantity of oil present; minimum concentration necessary for acceleration of corrosion has not been determined.—*R. E. Thompson.*

The Purification by Bacterial Decomposition of Gas Plant Effluent Containing Phenol. M. Kalabina and C. Rogovskaya. Z. Fischerei, **32**: 153–70, 1934; Wasser u. Abwasser, **32**: 124–5. From Chem. Abst., **28**: 4208, July 10, 1934. Rate at which phenol was destroyed when effluent was kept in open jars in laboratory was occasionally as high as 100 p.p.m. per day and averaged 57 p.p.m. per day. Phenol in concentration up to 3 grams per liter serves as source of carbon to certain bacteria. The higher the phenol concentration, the more sewage must be added. When sufficient minerals are present, rate of decomposition first increases and then decreases as phenol concentration is raised from 50 milligrams to 3 grams per liter. Optimum rate is at concentration of from 0.5 to 1 gram per liter. Temperature and aëration are important. —*R. E. Thompson.*

Must Ammonium Chloride be Added in Calcium Determinations? K. Scheringa. Chem. Weekblad, **30**: 598, 1933. From Chem. Abst., **28**: 4333, July 20, 1934. Ammonium chloride is unnecessary in presence of aluminum; latter can be separated by neutralizing solution to phenolphthalein and filtering with filter aid. In presence of magnesium it delays precipitation of magnesium oxalate, but this can be achieved by using excess oxalic acid. Solubility of magnesium oxalate is 1 in 1800 in water, 1 in 700 in saturated ammonium chloride, and 1 in 500 in saturated ammonium oxalate. When magnesium is to be determined with soap, all ammonium salts must be avoided and potassium oxalate employed.—*R. E. Thompson.*

Improving the Taste of River Water by the Use of Activated Charcoal. A. F. Meyer and A. van Rossen. Water, No. 19, 1933; Wasser u. Abwasser **31**: 361. From Chem. Abst., **28**: 4508, July 20, 1934. Granular activated carbon retained its effectiveness for short time only and was too expensive. Better results were obtained by adding 10 p.p.m. powdered activated carbon to water before passing to settling basins. After 10 hours, water was slow-filtered. Only small amount of carbon reached filter and this was retained by first 4 centimeters of sand.—*R. E. Thompson.*

Anomalies Observed in the Rate of Corrosion of Zinc. J. E. Maconachie. Trans. Electrochem. Soc., **66**: 8 pp. (preprint), 1934. From Chem. Abst., **28**: 4361, July 20, 1934. Investigation of corrosion of galvanized iron hot water storage tanks. Two anomalies were found in rate of corrosion of zinc in distilled water: presence of maximum in loss of weight with time curves at temperatures above 50°; and occurrence of specimens which showed marked passivity. Apparatus and technic which give large amount of reproducible data are described. Initial rate of corrosion of zinc in distilled water is greatly increased by increasing temperature up to about 60°, above which rate decreases. Corrosion rate at 60° appears subject to anomalous fluctuation which can only be explained on basis of reversal of corrosion reaction. There appears to be surface condition, not affected by pickling for half a minute in normal hydrochloric acid, which causes zinc to be extraordinarily resistant to attack by distilled water. Occurrence of this condition is infrequent in fresh sheet, but it appears to considerable extent after aging. Individual specimens cut from

one sheet show differences in susceptibility to corrosion which appear to be independent of method of measurement.—*R. E. Thompson.*

The Detection of Indole in Bacterial Cultures by Means of *o*-Nitrobenzaldehyde. BENIAMINO JOLLES. Diagnostica tec. lab. (Napoli) Riv. mensile, **5**: 8–14, 1934. From Chem. Abst., **28**: 4445, July 20, 1934. This reagent is more specific than that of EHRLICH and color produced is more stable.—*R. E. Thompson.*

Light-Electric Colorimetry in Water Analysis. E. NAUMANN and K. NAUMANN. Z. anal. Chem., **97**: 81–6, 1934. From Chem. Abst., **28**: 4507, July 20, 1934. Experiments with LANGE'S colorimeter show that instrument can be used to advantage in colorimetric and nephelometric testing of water. Examples are the determinations of iron, manganese, phenols, sulfate, and lead. By increasing depth of liquid observed, accuracy is increased.—*R. E. Thompson.*

The Bacteriological Action of a Norite Filter in Household Use. L. H. LOUWE KOOYMANS and J. A. WIEGAND. Zentr. ges. Hyg. Bakt. Immunitäts, 30: 599–600, 1934; Wasser u. Abwasser, **32**: 105. From Chem. Abst., **28**: 4508, July 20, 1934. Filtered water had lower count than untreated, but reduction was not sufficient to render polluted water potable. No selective action toward pathogens was noted.—*R. E. Thompson.*

Determination of Trisodium Phosphate in Treated Waters. V. TABAKOFF. Industrie chimique, **20**: 806, 1933. From Chem. Abst., **28**: 4508, July 20, 1934. Method given below, which depends on formation of blue compound by treatment with stannous chloride, molybdate, and sulfuric acid, serves for determination of from 0.5 to 2.5 p.p.m. P_2O_5. To 10 cc. sample add 5 drops of mixture of 10 per cent ammonium molybdate solution with half its weight of concentrated sulfuric acid, 8 drops freshly prepared solution containing 0.2 to 0.5 gram tin, and 1 or 2 drops 0.1 per cent copper sulfate solution in 2 normal hydrochloric acid. Compare with standards containing known amount of phosphate.—*R. E. Thompson.*

Water Solubility of Copper with Respect to Its Use in Water Systems. VICTOR FROBOESE. Gas- u. Wasserfach, **77**: 225–31, 1934. From Chem. Abst., **28**: 4508, July 20, 1934. Lowest concentration of copper at which it can normally be tasted is 1.5 p.p.m. Carbon dioxide increased solubility of copper when normal oxygen content was present. Cuprous oxide films such as are present in new tubing are immediately attacked by carbon dioxide. Increased sodium chloride or magnesium sulfate content did not increase rate of corrosion, and increased calcium content decreased it. Favorable conditions are low oxygen content, lowest possible carbon dioxide, and at least average carbonate hardness.—*R. E. Thompson.*

Chlorine Sterilization and Some of Its Problems. H. W. LEHMKUHL. Ann. Rept. New York State Assoc. of Dairy and Milk Inspectors, **6**: 15–36, 1932.

From Chem. Abst., 28: 4508, July 20, 1934. Relationship of active chlorine, total chlorine, and pH to germicidal efficiency of solutions of hypochlorites, chloramine-T, and other chlorine-bearing organic compounds is discussed. It is important to determine effects of pH on relative rates of chlorination and oxidation by chlorine.—*R. E. Thompson.*

The Equilibrium between Carbonate Hardness and Free Carbonic Acid in Natural Waters. A. EMUNDS. Chem.-Ztg., 58: 328, 1934. From Chem. Abst. 28: 4508, July 20, 1934. Neither formula nor curve given by TILLMANS and HEUBLEIN (cf. C. A., 6: 3301) for equilibrium between free and combined carbon dioxide in calcium bicarbonate solution holds in presence of other bicarbonates or of other calcium and magnesium salts. Salts differ in their effects, so that no generalization is offered.—*R. E. Thompson.*

New Precipitation Methods of Softening Water. L. LIST and J. LEICK. Wärme, 56: 752–5, 1933; Wasser u. Abwasser 32: 48. From Chem. Abst., 28: 4509, July 20, 1934. Carbonate was removed 3 times faster when quartz sand was present. Calcium carbonate also catalyzed softening process. Through choice of suitable catalysts and equipment, time for softening was reduced from between 2 and 3 hours to between 7 and 10 minutes.—*R. E. Thompson.*

The Softening of Boiler Water. MARTINY. Maschinenschaden, no. 4, 61–5, 1933; Wasser u. Abwasser, 31: 364. From Chem. Abst., 28: 4509, July 20, 1934. Carbonate and phosphate methods are compared. Danger of silicate scale formation in soda process is emphasized. Author recommends addition of carbonate until hardness of 1 to 2° (German) remains and then addition of sodium triphosphate until slight excess of about 5 grams per cubic meter is reached.—*R. E. Thompson.*

"Water Bloom" as a Cause of Poisoning in Domestic Animals. C. P. FITCH, LUCILLE M. BISHOP, W. L. BOYD, R. A. GORTNER, C. F. ROGERS and JOSEPHINE E. TILDEN. Cornell Veterinarian, 24: 30–9, 1934. From Chem. Abst., 28: 4509, July 20, 1934. Phenomenon known as "water bloom" occurs when waters of lake suddenly become turbid from growth of blue-green algae. Cattle, sheep, hogs, horses, and fowls have recently been fatally poisoned by drinking such waters from Minnesota lakes. Analyses of the algae gave negative results for poisonous metals, cyanides, and alkaloids. With guinea pigs as indicators of presence and potency of toxic material, it was found that poison is non-volatile; is heat-stable in the dry material (102–104° for 100 hours); is heat-stable in wet material when heated to 100° for any appreciable period, unless algae have been freshly taken from lake; remains potent when air-dried; can exist in water around the algae, for liquid separated from algae is toxic; does not migrate under imposed electrical potential; is of small molecular weight; disappears on putrefaction of algae; is sometimes resistant to acidic or alkaline conditions somewhat removed from acidity of water in which it is produced; does not react chemically as a toxalbumin; and is not a botulinus toxin. All tests indicate that poison is an organic compound.—*R. E. Thompson.*

The Use of Sodium Aluminate in Boiler Water Treatment. H. N. BASSETT. Steam. Engr., 3: 321–2, 1934. From Chem. Abst., 28: 4509, July 20, 1934. Disadvantages of lime-soda process discussed and value of sodium aluminate in overcoming these drawbacks described.—*R. E. Thompson.*

Determination of Iron and Copper by Means of Hematoxylin. S. RIVAS GODAY. Bol. farm. mil. 11: 369–73, 1933; Chimie industrie, 31: 1056. From Chem. Abst., 28: 4680, August 10, 1934. Description of color reactions of ferrous and ferric iron and copper with hematoxylin and of colorimetric determination of copper.—*R. E. Thompson.*

A Formulation of Bacterial Changes Occurring in Polluted Water. H. W. STREETER. Sewage Works J., 6: 208–233, 1934. From Chem. Abst., 28: 4510, July 20, 1934. Comparative mathematical study of curves defined by progressive changes in bacterial content observed in Ohio River below Cincinnati and in samples of river water stored in laboratory for 40 days or longer at 10°, 20°, and 37° indicates that rates of bacterial decline observed in stored samples and likewise in river itself, when values for latter are corrected for sedimentation effect, represent fair approximation to true bacterial death rates at a particular temperature and under conditions of natural purification in a stream. Two empirical equations have been developed from data to express trend of bacterial curves derived from stored sample tests and river observations. Constants derived from several typical bacterial curves bear definite relation to temperature and to relative density and age of pollution. From comparative study of empirical equation with others developed from simple assumptions, it is indicated that former are shaped primarily by influence of active suppressive force which probably represents effect of bacteria-consuming plankton. Equations are compared with those derived by other observers for similar phenomena. Under conditions of laboratory test for biochemical oxygen demand, total demand satisfied up to any time appears to bear definite relation to cumulative numbers of bacteria present in sample up to that time.—*R. E. Thompson.*

Phenols in Highway Tars and Their Solubilities in Water. LOUIS SCHUMANN. Chem. Obzor., 9: 23–7 (in English 27), 1934. From Chem. Abst., 28: 4856, August 10, 1934. Properly prepared tar contains only about 0.04, 0.17 and 0.21 per cent, respectively, of phenol, cresols, and xylenols. At 20°, solubility of phenol is 8.5 per cent; that of each of 3 cresols 2.5 grams per 100 cc.; that of xylenols, α- and β-naphthol, and anthranols, respectively, 1.5, 0.21, 0.06, and 1.25. Distribution ratios for each between water and benzene and water and tar were determined. Experiments indicate that rain water remaining for short time in contact with tar surfaced roads cannot extract phenols in amount sufficient to injure fish when drainage runs off into river.—*R. E. Thompson.*

Metallic Coatings as Protective Media. S. ROBSON and P. S. LEWIS. Chemistry and Industry, 54: 26, 605–616, June 28, 1935. Types of corrosion are described and metallic coatings of zinc, aluminium, tin, cadmium, nickel, etc.

are discussed. In section dealing with corrosion by water, it is stated that galvanized pipes for water are not harmful. There is no legal limit for zinc in England, but in certain of United States, 5 parts per million is legal limit. Drinker, Fehnel and Marsh (J. Biol. Chem., **82**: 377) state that no upper limit can be set from toxicity standpoint and that at all concentrations which can be ingested without emesis they observe no harmful effects. At 25-30 parts of zinc per million, astringent metallic taste is so strong that water is rejected as unpalatable. In J.A.W.W.S., **26**: 55, it was tentatively suggested that 40 parts of zinc per million be taken as safe upper limit in drinking water. Tin lining of lead pipes conduces to safety, but lining should not be of tin-lead alloy. Copper may be used for all but acid waters, when it should be tin-lined. Copper oxide is less soluble and less toxic than lead oxide and is not cumulative in system. In section on soil corrosion it is stated that galvanizing is generally superior to lead coating for external protection of pipes. Soil corrosion is caused to large extent by localized electro-chemical action set up by oxygen concentration cells, by presence of mill scale, or by differences in soil contacts. Galvanic currents due to pipe lines passing through different soils may also cause corrosion. Oxygen diffusion may be lessened by deeper burial of pipes in compact soil.—*W. G. Carey.*

Absorptive Properties of Synthetic Resins. Part 1. B. A. Adams and E. L. Holmes. Chemistry and Industry, **54**: 2, 1-6 T, January 11, 1935. Resins made by action of formaldehyde on tannins, e.g. quebracho, wattle bark, Indian acacia, cutch, etc. have base-exchange softening properties comparing favorably with glauconite. Acid-calcium exchange may be made, giving filtrate containing no dissolved salts, but original anions as free acids. Since resins remove completely alkalis and carbonates from solution they may be used for removal of lime in "excess lime" water treatment, and for removal of excess of lime and of sodium carbonate, as well as of last traces of calcium and magnesium from lime-soda softened water. Removal of ammonium salts, iron, manganese, lead, copper, and zinc can also be effected. By consecutive use of phenolic and amino resins, it is possible to remove completely dissolved salts, leaving filtrate equivalent to distilled water. This was proved upon tap water, by using first quebracho tannin resin to remove cations, and then a meta-phenylenediamine resin to remove anions; total solids being reduced from 33 parts to about 1 part per 100,000.—*W. G. Carey.*

The Rôle of Sodium Aluminate in Accelerating the Separation of Solid Phases during Water Softening Operations. Part II. The Effect on Mixed Solutions of Calcium and Magnesium Salts. L. M. Clark and P. Hamer. Chemistry and Industry, **54**: 4, 25-28 T, January 25, 1935. Mechanism of sodium aluminate coagulation was discussed in Part I (see J.A.W.W.A. **26**: 4, 561, April 1934). Sodium aluminate is effective in removing magnesium when amount is as little as 2 parts per 100,000, whether as temporary or permanent hardness. Softening of calcium chloride by sodium carbonate is more effective with excess of caustic alkalinity, provided by calcium hydroxide and sodium carbonate rather than by sodium hydroxide. This effect is ascribed to inducement of crystallisation by nuclei of calcium carbonate provided by reaction between

solid phase of calcium hydroxide and sodium carbonate. Similar effect is noted when calcium hydroxide precipitates calcium bicarbonate, for in such a reaction, rate of precipitation is greater than when sodium carbonate reacts with calcium chloride solution.—*W. G. Carey.*

The Rôle of Sodium Aluminate in Water Softening. Part III. Factors Influencing the Capacity and Efficiency of Lime-Sodium-Carbonate Water-Softening Plants. L. M. CLARK and W. R. COUSINS. Chemistry and Industry, 54: 21, 143–149 T, May 24, 1935. At all temperatures, sodium aluminate reduced residual hardness of lime-soda softened water, and at each temperature there was definite quantity of aluminate, further increase above which showed no decrease in hardness; this minimum quantity of aluminate decreased with rise of temperature. Water with hardness of 1 part per 100,000 could be obtained by use of aluminate at 60° and no improvement was found at 90°. Increase in rate of flow of water through settler had only small effect on residual hardness until critical value reached, when hardness rose rapidly; but it was possible to exceed this critical rate by 25 per cent before hardness rose to figure obtained at low rates of flow without aluminate. Addition of sodium aluminate at point subsequent to lime and soda addition showed no advantage over mixing aluminate with other softening agents.—*W. G. Carey.*

Sodium Aluminate in Filter Plant Practice. Anon. Chemistry and industry, 54: 4, 78–80, January 25, 1935. Among disadvantages attending use of aluminum sulfate in water, relying on temporary hardness to provide alkalinity for floc formation, are: (1) aluminum sulfate reacts slowly with calcium bicarbonate, possibly in three stages, and cases may arise where insufficient reaction time may result in presence of basic aluminum sulfate in solution, recognizable as "residual alumina"; (2) carbon dioxide formed by interaction of alum and bicarbonate increases corrosiveness and depresses pH, possibly below the zone for optimum coagulation; and (3) temporary hardness is converted to permanent hardness in direct proportion to amount of alum added. Sodium aluminate provides both alumina and alkalinity necessary for floc formation, containing about 3.5 times as much soluble alumina as strongest filter alum, and sodium alkalinity corresponding to about 40 per cent content of Na_2O. This latter reacts with free carbon dioxide and gives coarse and well defined floc, with no conversion of temporary to permanent hardness. It speeds up reaction and gives wider pH range for optimum coagulation, and reduces alum requirement, while improved sedimentation, due to coarser floc, lightens load on filters. Typical examples of its use in English purification plants are given. —*W. G. Carey.*

Rapid Determination of Free Chlorine in Water. L. LEROUX. Compt. rend. 1934, 199: 1225–1227. From Analyst, 60: 707, 113, February 1935. Chlorine in amounts less than 0.5 parts per million may be estimated by shaking 50 cc. of the water with a crystal of potassium bromide, adding 1 cc. of magenta (fuchsine) solution (10 cc. of 0.1 per cent aqueous magenta solution with 100 cc. of 5 per cent sulfuric acid) and 1 cc. of acetic acid. Violet color reaches maximum in 15 minutes and is compared with standard chlorine solution simi-

larly treated, or, in the field, with drops of N/500 permanganate, color given by permanganate having been previously standardised against chlorine by above method. Test is unaffected by organic matter (urea) or nitrates.—*W. G. Carey.*

The Determination of Zinc in Water by Means of Sodium Diethyldithiocarbamate. W. R. G. Atkins. Analyst, 60: 711, 400–401, June 1935. Zinc may be estimated turbidimetrically in water, in concentrations of from 0.05 to 25 parts per million, using a 0.1 per cent aqueous solution of sodium diethyldithiocarbamate and comparing turbidity with that given by zinc sulphate standards with reagent, in Nessler cylinders. Unlike lead, turbidity disappears in ammoniacal solution. Iron may be removed previously with ammonia. Method is as sensitive as ferrocyanide and is more convenient than resorcinol, which is affected by calcium and requires time for development of blue color.—*W. G. Carey.*

Volumetric Determination of Calcium and Magnesium in Water (Micro Method). A. J. Amiantov. J. Appl. Chem. Russ., 1934: 7, 632–635. From Chemistry and Industry, 54: 2, 48 B, January 11, 1935. Calcium is precipitated from 25 cc. of water as oxalate, collected and washed on centrifuge, and determined by permanganate. Magnesium is determined in centrifugate by adding 5 drops each of 25 per cent ammonia, and of ammonium arsenate. Precipitate is washed with 0.5 per cent ammonia after which 5 cc. of 20 per cent sulfuric acid and 5 drops of 50 per cent potassium iodide are added. One hour later, liberated iodine is titrated with 0.005 *N* sodium thiosulfate, 1 cc. of which equals 0.0001 gram MgO.—*W. G. Carey.*

The Watlington Water Works of Bermuda. W. D. Turner. Chemistry and Industry, 53: 40, 819–823, October 5, 1934. Until recently, fresh water supply was rain water collected from roofs, and water imported from New York. Vertical drillings pass through porous coral sandstone, then through salt water and into volcanic rock without striking fresh water. Annual rainfall is 40–60 inches and to catch this fresh water as it percolates through porous coral, but before it reaches salt water level, an infiltration trench 400 feet long, lined with tile, and about 18 inches above sea water level was excavated. Site chosen was relatively narrow valley served by large area for run-off. Supply, under present operating conditions is 7 million gallons per month of pure, fresh water with 250 parts per million hardness, 80 parts per million sodium chloride, and *B. coli* content of less than one in 10 cc. Water is softened with lime and 1.2 grains per gallon sodium aluminate to 40 parts per million of hardness and is then passed through slow sand filters. Chlorination, used as precautionary measure, is effected with hypochlorite from one single electrolytic cell, and water is delivered through asbestos mains.—*W. G. Carey.*

Paris Water Supply. Anon. The Engineer, 159: 4129, 218–219, March 1, 1935. Large part of supply of 178 million gallons daily is obtained from 50 springs within radius of 100 miles, water being conveyed through 5 aqueducts having total length of 383 miles to 3 reservoirs. This supply is supplemented

by possible 144 million gallons of river water from Seine and Marne and from Ourcq canal. Water is filtered and chlorinated, except portion which is treated with ozone. Larger ozone plant (66 million gallons per day) is to be constructed: efficiency of ozone is unquestioned, and advantages of freedom from after taste are recognized, while cost of working has been reduced by recovery of excess ozone, and it is claimed that cost is little more than that of 20 watts of electricity per cubic meter of water treated, apart from capital and maintenance charges. Paris has Val de Loire water scheme in hand but completion will not be for 5–10 years. In this scheme, Loire water is to be filtered through natural beds of fine sand forming river's bank into porous concrete collector at depth of 20 feet and extending for some 25 miles along side the river.—*W. G. Carey.*

A Large Base Exchange Softening Plant. Water and Water Engineering, **37**: 448, 245–248, May 1935. Total capacity of Colne Valley (England) base exchange installation is 6¾ million gallons per day in two plants, softening from 21 degrees to 10 degrees. Flow is downward during softening and upward during regeneration, material used being "Doucil", a synthetic sodium aluminum silicate, having higher base exchange capacity than has natural greensand. Two-stage regeneration is employed, used brine from one unit serving as first charge in regenerating another, flushing water which follows being used for making new brine. Unit softening 200,000 gallons of 21° hardness to zero in 10 hours requires 16 hundredweight of salt for regeneration. Unsoftened water is added to bring final effluent to 10 degrees.—*W. G. Carey.*

Some Problems of Water Supply. A. PARKER. Chemistry and Industry, **54**: 3, 49–54, January 18, 1935. Estimates of quantity of water required must be planned for needs of thirty or forty years ahead. National water grid proposed in England would entail enormous expenditure without commensurate benefit. Problems of quality include improvement in river conditions as regards trade and sewage effluents. Treatment includes removal of suspended and colloidal substances, destruction and prevention of algal growth, removal of iron and manganese, reduction of hardness, improvement of color, odor, and taste, disinfection to remove harmful bacteria, and treatment to prevent plumbo-solvency and corrosive action. These various methods of treatment are enlarged upon, mention being made of softening by barium aluminate and by sodium phosphate. In connection with iodine deficiency, example set by Rochester, New York, in adding sodium iodide to water is cited. Reference is also made to fluorine excess in water.—*W. G. Carey.*

The Sterilization and Filtration of Water. Sterilization Methods and Experiments on Dirt Penetration. C. B. BRAMWELL. Water and Water Engineering, **37**: 445, 105–109, March 1935. Progress in sterilization with ozone has been retarded by high cost of electricity, but recent installations include Paris (19.8 million gallons per day), Nice (18), and Petrograd (11.4). Treatment is advantageous in reducing or removing suspended matter, color, odor, and taste and in giving sparkling, palatable water. Complete sterilization is claimed for catadyn treatment and effect continues for long periods; but it can

only be applied to clear bright colorless water. Same disadvantage applies to ultra-violet rays. Liquid chlorine has superseded use of bleaching powder, usual dose from 0.25–0.5 parts per million giving absence of *B. coli* in 100 cc., although some harmless, spore-forming bacilli can withstand up to 15 parts per million. Ammonia-chlorine treatment prevents aftergrowths, tastes, and odor in stored water. Penetration of dirt in filters was found to decrease rapidly towards bottom of filter and to depend on grade of material, the head, nature of impurities, and preliminary treatment and coagulation of water. In horizontal cylindrical filters with spherical ends and in those with conical sides, dirt accumulation was less at sides, while with straight-sided filters, it was greater. Diagram shows benefit of curved or conical sides, effective cleaning of which may be carried out by mechanical agitation.—*W. G. Carey.*

Sanitation and Water Purification. J. H. GARNER. Reports of the progress of applied chemistry (issued by Society of Chemical Industry, London) 19: 1934, 733–771. General opinion respecting rivers during drought of 1933–34 was that they withstood well severe tests of low flows and high temperatures; but in view of probable more extensive use of rivers for water supply, standards of purity of effluents should be investigated and reference is made to STREETER's (U. S. Pub. Health Reports, Reprint 1643, 1934) classification of water pollution problems in U. S. according to purposes for which stream water is required. Summary is given of experiences in America and elsewhere on control of algae in reservoirs by copper sulfate, effect of pH value on algal growth, and removal of copper by ferric floc. Reference is made to various coagulants, including alum containing 2 percent active carbon, as used in treatment of Delaware River at Trenton, N. J. Experience is cited of sodium aluminate treatment to improve flocculation and also to raise pH, obviating subsequent treatment with lime. Disinfection of water supplies by chlorine, and ammonia-chlorine, and use of metallic magnesium for dechlorinating are referred to, and attention is also given to catadyn treatment of water, effect of suspended matter on oligodynamic effect, and to clinical significance, removal, and determination of small quantities of fluorine in water supplies. BARR & THOROGOOD's method of fluorine determination is given (see J.A.W.W.A. 27: 9, 1253, 1935).—*W. G. Carey.*

The Use of Activated Carbon in the Purification of Water in the Tropics. (The Madras Water Supply). T. N. S. RAGHAVACHARI and P. V. SEETHARAMA IYER. Reprint from The Proceedings of the Indian Academy of Sciences, II, 3, 237–253, Sec. B, September, 1935. Ever since the slow sand filters have been in operation in Madras, tastes and odors have been consistently present in the filtered water, due to excessive organic pollution, production of hydrogen sulfide in the filters, etc. Extensive experiments on the use of activated carbon at a local industrial filter plant and at the experimental filter plant maintained under the Madras Government are described. The experiments included addition of powdered activated carbon to the water in the filters and the placing of a 1- or 1½-inch layer of granular activated carbon in the sand beds. The latter was found more effective in removing tastes and odors, color, and organic matter. Hydrogen sulfide was produced in minute traces only in

the filters containing carbon when operated at rate of 4 inches vertical per hour and not at all when rate of filtration was doubled. Low rates of filtration have always been found conducive to hydrogen sulfide production. The control unit, containing sand only, produced, as usual, large amounts of hydrogen sulfide. It was also found that passage of slow sand filtered water containing large and varying amounts of hydrogen sulfide and white filamentous growths through 2-foot layer of granular carbon, allowing contact period of 36 minutes, rendered the water esthetically perfect (sparkling, clear, colorless, odorless, and tasteless) and reduced the organic matter content 75 per cent. The granular carbon has shown no evidence of deterioration in quality after 23 months' continuous use, although subjected to only the usual cleaning when the sand filters became clogged. An estimate is included of the cost of carbon treatment as employed. In the tabulations of results, a scheme developed by H. H. KING is used, in which numerical values are assigned to the more important results. Bibliography of 28 references appended.—*R. E. Thompson.*

Adaptation of Venturi Flumes to Flow Measurements in Conduits. H. K. PALMER and F. D. BOWLUS. Proc. Amer. Soc. Civil Engineers, **61**: 7, 961, September, 1935. Weirs may be undesirable as measuring devices in conduits because of ponding effect on upstream side, changes in weir coefficients, and losses of head, and for other reasons. Occasionally, Venturi flume has been used to overcome these disadvantages. This paper presents the theoretical hydraulic principles involved, and the results of special tests made, in the adaptation and construction of various Venturi flumes for measuring flow in conduits of uniform cross section where weirs have proved unsatisfactory. Proper functioning of Venturi flume requires that parallel flow occur in the channel above the flume and in the throat. Necessary critical velocity is obtained only when drop in energy head occurs just below the throat. A small jump in the water surface at this point is positive evidence that critical velocity is occurring. In addition, throat must have sufficient length, or else water will not be flowing in parallel filaments at point of critical depth. Ideal flume throat would have such size and shape that ratio of the cross-section of the water in the throat to that in conduit would be the same for all quantities. For circular pipes it has generally been found desirable to install a bottom slab in addition to the two sides. Any formula for flow in open channels may be used for first construction of quantity-depth curve, but a different procedure is suggested. By superimposing the quantity-energy-head curves for the throat upon those for the pipe and then moving the throat tracing up or down, it is possible to decide quickly which throat size should be used and how high it should be set above the invert. Placing the center of the throat at about the lower side of a manhole gave proper spacing for installing the stage-recorder float near the upper side, at a point where flow in the pipe was uniform. Repeated checks of indicated stage records against actual depth measurements show no appreciable error.—*H. E. Babbitt.*

The Stress Function and Photo-Elasticity Applied to Dams. J. H. A. BRAHTZ. Proc. Amer. Soc. Civil Engineers, **61**: 7, 983, September, 1935. Object of Part I of this paper is to familiarize engineers with use of the AIRY

stress function for solution of problems in plane stress and plane strain when ordinary engineering methods fail to give even approximate results. Object of Part II is to familiarize the engineer with the photo-elastic phenomenon and its application to civil engineering structures and to compare results obtained in this way with the theoretical results obtained in Part I. In Part I the stress function is defined in rectangular and polar coördinates, convenient forms of boundary conditions are treated, and various applications of the theory are developed. For example, in order to indicate the general procedure for determining the stress functions in the case of 2-dimensional structures for a given loading, the case of the infinite wedge with hydrostatic pressure on one side, and no body forces, is developed in some detail. It is shown that all stresses are linear along any straight line in the triangular dam of infinite height, but not so near the base of a dam of finite height. The second application is to foundation stresses. The third is to stresses in the region of the base of the gravity dam. The fourth to approximate boundary stresses at sharp and rounded corners or fillets. Examples are computed by approximate theory. In Part II, the photo-elastic experiments in connection with the Morris Dam in California are explained and the results of actual measurements are reported and analyzed.—*H. E. Babbitt.*

Tunnel and Penstock Tests at Chelan Station, Washington. E. R. Fosdick. Proc. Amer. Soc. Civil Engineers, **61**: 8, 1131. October, 1935. An analysis of flow line losses was computed from data obtained by tests on the unusually long flow line leading to hydro-electric station at Chelan, Wash. Total head loss is 400 feet in 10,478-foot length of conduit, 14 feet in diameter, including sections of concrete, and of riveted steel, together with a Y-branch, and a surge tank. Entrance losses were found to be small and to vary as Q^2. Losses in the concrete were found to correspond with those obtainable from Hazen-Williams formula by use of coefficient of 134. Losses in steel penstock were found to vary with $Q^{1.75}$, (corresponding to variation in Hazen-Williams coefficient from 82 to 96, depending on the velocity. Losses in Y-branch were found to be relatively high. Diversion losses in lower penstock bends were found to vary with the 2.20 power of the flow, while those in the Y-branch were found to vary with the 1.75 power.—*H. E. Babbitt.*

Water-Bearing Members of Articulated Buttress Dams. H. D. Birke. Trans. Amer. Soc. Civil Engineers, 100: 55, 1935. Equations developed are sufficient for precise determination of most economical proportions for decks and haunches of articulated buttress dams. Most economical relation between span of deck and clear buttress spacing is shown to be independent of the buttress spacing for a given height of dam. Quantity of concrete and steel in decks and haunches, as well as total cost of these members, is shown to be in direct proportion to the square of the buttress spacing. Curves given may be applied to actual design problems. Fortunately, the flexible, articulated buttress dam is subject to an exact mathematical analysis both as to economy as to stresses, since there are no statically indeterminate elements introducing mathematical and structural uncertainties.—*H. E. Babbitt.*

Model of Calderwood Arch Dam. A. V. Karpov and R. L. Templin. Trans. Amer. Soc. Civil Engineers, **100**: 185, 1935. Writers present a discussion of theoretical principles on which model study is based, of how these principles are incorporated in the model, and of results of a series of tests made on the model in conjunction with some of the tests made on the prototype. As a result of the tests, following conclusions may be drawn: it is unsafe to assume a non-yielding foundation in design; assumption of straight-line stress distribution will result in an inadequate design; assumption of a monolithic structure is inadequate, because of bending moments in the arches; circular arches produce non-uniform horizontal stresses, which can be avoided by properly-shaped arches; gravity action should not be neglected, for economy, in design; the reduction of vertical tension on both faces merits attention; the adopted rubber compound is suitable for constructing models; it will repeat its behavior under loadings; it will give high sensitivity and large distortions; displacements should be measured in three planes; and the methods devised for measuring strains proved satisfactory.—*H. E. Babbitt.*

Formation of Floc by Ferric Coagulants. E. Bartow, A. P. Black, and W. E. Sansbury. Trans. Amer. Soc. Civil Engineers, **100**: 263, 1935. Ferrous sulphate, ferric chloride, chlorinated copperas, and ferric sulphate are used as coagulants in water purification. Natural waters contain varying quantities of sulfate ion, chloride ion, sodium ion, calcium ion, etc. An experimental determination of the action of these ions on the formation of floc with ferric salts at different pH values shows the following results: (1) on the acid side, the sulfate ion has much greater effect on coagulation than the chloride ion; (2) using from 25 to 250 p.p.m. of sulfate ion, there is little change in the effect produced; (3) between a pH value of 6.5 and of 8.5 there is a zone in which ferric floc forms slowly, or not at all; and (4) in and beyond this zone, sodium and calcium ions are most effective in coagulation. Assumption of change in sign of charge on colloidal ferric floc from positive, where it is more affected by sulfate or chloride ions, to negative, will explain the zone of no floc formation and the more effective action of sodium and calcium ions beyond this zone. The quantity of residual iron in solution is roughly proportional to the time required for the floc to form.—*H. E. Babbitt.*

An Approach to Determinate Stream Flow. M. M. Bernard. Trans. Amer. Soc. Civil Engineers, **100**: 347, 1935. Method for determination of stream flow from rainfall records is presented, based upon unit graph constructed by segregating daily stream flow resulting from a single downpour of one day from flow occasioned by previous and subsequent rainfall. This is then converted into run-off in c.f.s., equivalent to a depth of 1 inch over the area. Method requires the construction of a distribution-graph based upon climatological records. Graph is found to be a function of the watershed characteristics. The "pluviagraph" is discussed and its value to design in the field of hydraulic engineering is demonstrated.—*H. E. Babbitt.*

Discharge Formulas and Tables for Sharp-Crested Suppressed Weirs. C. G. Cline. Trans. Amer. Soc. Civil Engineers, **100**: 396, 1935. Following em-

pirically deduced discharge formula applies to sharp-crested weirs that occupy full width of the channel:

$$Q = L(3.276 \times (1.0195)^H \times H^f \times 10^{KH})$$

in which, Q is the discharge in c.f.s.; L is the length of the weir crest in feet; H is the head on the weir, in feet; $f = 1.5064H^{0.0081}$ and K is a function of the height of the weir. A discharge table is given in c.f.s. for heads up to 3 feet and for various heights of weir.—*H. E. Babbitt.*

Rainfall Studies for New York, N. Y. S. D. Bleich. Trans. Amer. Soc. Civil Engineers, **100**: 609, 1935. Formulas and graphs have been derived from a collection of rainfall data in New York City. Records of heavy rainfalls in Central Park are given from 1869 to 1930. Six formulas of the modified exponential type, $R = \frac{C}{(t+d)^e}$ are given for various time intervals from one to 50 years; in which R is the intensity of rainfall, C, d, and e are empirical constants, and t is the time of duration of the storm. The general formula $Rn^{0.3} = \frac{42.5}{(t+12)^{0.85}}$, does not appear sufficiently close to the observations to warrant its use for all kinds of storms. In this formula R is in inches per hour, t is in minutes, and n is the average number of times per year that this rate occurs, or is exceeded.—*H. E. Babbitt.*

The Reservoir as a Flood-Control Structure. G. R. Clemens. Trans. Amer. Soc. Civil Engineers, **100**: 879, 1935. Operation of the flood-control reservoir for various other purposes, such as water power, irrigation, navigation, and water supply, combined with flood control, is considered. No combined project is possible, unless storage is more than is required for flood control and unless combined operation of separate storage zones is possible. In effect this means a flood surcharge on a water supply. The reservoir has many disadvantages as a flood-control structure: it is difficult to locate and to operate. Nevertheless, advantages of protecting an area by reducing the flood crest that reaches it are such, that a reservoir solution should be analyzed carefully before any flood-control plan is adopted.—*H. E. Babbitt.*

Flow of Water Around Bends in Pipes. D. L. Yarnell and F. A. Nagler. Trans. Amer. Soc. Civil Engineers, **100**: 1018, 1935. Presents more outstanding results of series of experiments on flow of water around bends of various shapes and of various degrees of curvature in 6-inch pipes. It is concluded that: (1) all bends result in additional loss of head; (2) the velocity filament along inside wall of bend is increased and along outside wall is decreased; (3) for any given pipe bend and given rate of flow, head lost depends upon velocity distribution in the approach channel; (4) under certain conditions, loss of head in the bend may be four times as much as would occur in same bend under normal conditions; (5) it is possible to compute mean velocity and discharge in pipe from knowledge of maximum difference in pressure between inside and outside of bend; (6) a calibrated pipe bend can be used as a meter; (7) losses

in bends appear to vary as the square of the velocity; and (8) fundamental laws that apply to closed conduit bends also apply to open channel bends.—*H. E. Babbitt.*

Security From Under-Seepage beneath Masonry Dams on Earth Foundations. E. W. LANE. Trans. Amer. Soc. Civil Engineers, **100**: 1235, 1935. Results of an investigation of more than 200 masonry dams with various kinds of earth foundation, to determine length of percolation path necessary to prevent failure from under-seepage, or piping. Extensive tabulations are given of data for these dams. Included among conclusions reached are: (1) ordinary method of analyzing masonry dams on earth foundations to secure safety against piping is faulty; (2) piping may occur in two ways; by flow along line of contact of structure with its foundation, or by flow directly through foundation material; (3) resistance to piping along contact surfaces inclined at less than 45 degrees to the horizon should be considered as only one-third of that offered with slopes of 45 degrees or more (it is to be noted that these slopes are those of the surface of contact and that slope taken by the water may be different); and (4) the weighted-creep distance of a cross-section of a dam is the sum of the vertical creep distances plus one-third the sum of the horizontal creep distances, and the weighted-creep:head ratio is the weighted creep divided by the effective head. Reverse filters, weep-holes, and drains are aids to security, and weighted-creep ratios may be reduced as much as 10 per cent if they are used. In order to prevent failure of dams by percolation directly through the foundation material, the short-path:head ratios should be not less than 0.8 of those recommended for the weighted creep.—*H. E. Babbitt.*

Uplift and Seepage Under Dams on Sand. L. F. HARZA. Trans. Amer. Soc. Civil Engineers, **100**: 1352, 1935. Basic principles of flow under dams on sand are presented. Analytical methods applicable to some familiar types of foundations, and graphical and electrical analogy methods of general application are described, by which can be determined the theoretical laws governing: (1) hydrostatic pressure along the foundation contact; (2) hydraulic gradient with which the water escapes upward to the toe; and (3) approximate leakage under the structure. Analysis indicates that, in general, material under a dam is in stable condition, with forces having a safe component downward, until tail water is approached, at which point the drop of internal pressure has a component upward, becoming vertical at the surface. The more rapid the upward reduction in pressure, the less becomes the effective weight of the material, until at a critical value the material floats away, with resulting rapid crumbling from the toe backward under the dam, causing failure.—*H. E. Babbitt.*

Air Conditioning and the Water Supply. Anon. The Technology Review, **38**: 1, 11, October 1935. Of late, the rapid spawning of air-conditioning and refrigerating machinery has added a new burden to the water supply engineer. Under the very best systems of compressor refrigeration, steam ejector, or absorption systems, 1200 to 3000 gallons of water would be required for a cooling period of 10 hours per day. Water-consumption index lags approximately

a year behind the more sensitive barometers of industrial production.—*H. E. Babbitt.*

Influence of Screen Capacity on the Yield of Wells. Anon. Johnson Nat'l. Drillers' Journal, 7: 3, 1, May-June, 1935. Article gives the details of tests on two 10-inch wells, spaced 12 feet 4 inches apart, at Osakis, Minn. Wells were identical, except for screens used. Tests indicate, besides other things, that in coarse, water-bearing formation, the yield of a well varies approximately in same proportion as the intake opening area of the screen.—*H. E. Babbitt.*

Dressing Drill Bits with Mechanical Dressers. Anon. Johnson Nat'l. Drillers Journal, 7: 4, 1, July-August, 1935. Discusses mechanical devices for dressing drill bits by power, as well as special shapes of drills.—*H. E. Babbitt.*

Reclaiming an Old Well and Reducing the Iron Content. R. M. Bragg. Johnson Nat'l. Drillers' Journal, 7: 4, 4. July-August, 1935. Geologist's account of reclaiming of 12-inch well, 22 years old. As result of study of geological conditions, well was deepened, flow was increased, and iron content was reduced.—*H. E. Babbitt.*

Advanced Methods of Pump Testing. Anon. Southwest Water Works Jour., 17: 8, 14, 1935. More accurate recordings of pump characteristics benefit the user of pumping equipment.—*O. M. Smith.*

Training School for Plant Operators and Repair Crews. L. C. Billings. Southwest Water Works Jour., 17: 9, 9–10, 1935. School course should include instruction along all lines of activity of the individual in conducting the business of furnishing a satisfactory supply of water and should be given by practical instructors.—*O. M. Smith.*

The Chlorine-Ammonia Treatment at Houston, Texas. L. P. White. Southwest Water Works Jour., 17: 9, 13–15, 1935. Chlorine added, contrary to the usual practice, before the ammonia gave the best results. The distribution of nitrogen in water with just no residual chlorine followed one of three changes: (1) free ammonia and albuminoid nitrogen increased; (2) free ammonia decreased, albuminoid nitrogen increased, and sometimes nitrites occurred; or (3) free ammonia decreased, with formation of nitrites and nitrates. Reducing the ratio of ammonia to chlorine from 1:4.5 to 1:60 increased residuals and decreased hydrogen sulfide odors.—*O. M. Smith (Courtesy Chem. Abst.).*

Methods for Handling Emergency Main Repairs. Henry E. Nunn. Southwest Water Works Jour., 17: 9, 19, 1935. Have an organization that is able to function promptly and efficiently at all times; have up to date and accurate records of the system; know the system and keep workmen who do; and keep up maintenance.—*O. M. Smith.*

Comprehensive Training Program Being Developed for Oklahoma Water and Sewer Operators. EDWARD R. STAPLEY. Southwest Water Works Jour., **17**: 9, 22, 1935. The proposed training plan, covering the basic sciences and the standard operating procedures, will require five years to complete.—*O. M. Smith.*

Factors Influencing Plumbosolvency. KARL HÖLL. Gesundheitsingenieur, **58**: 22, 323, June 1935. Experiments were conducted under close approach to tap conditions, by filling lead pipe with water under test and leaving under water pressure. Maximum values for lead content were attained after standing for at most three hours. Chloride ion concentrations of less than 500 p.p.m. showed no appreciable influence; with amounts up to 1000 p.p.m., lead content increased by from 10 to 20 per cent; with concentrations essentially exceeding 1000 p.p.m., increases of 50 per cent and even more in lead content were noted. Nitrites up to 30 p.p.m. of N_2O_3 failed to exert any effect on lead content; only with concentrations exceeding 50 p.p.m. of N_2O_3, associated with pH values below 7.0, did plumbosolvency increase. Nitrates not exceeding 100 p.p.m. of N_2O_5 and sulphates up to 200 p.p.m. of SO_4 also proved ineffective. It is concluded that under usual tap water conditions, oxygen content and pH value, or relation between free and combined carbon dioxide, are of primary importance. When used for earthing of radio apparatus, neither measurable currents nor noticeable internal corrosion were observed in lead pipe, except where old, poorly insulated batteries were in use.—*Manz.*

Experiences with Katadyn and Electrokatadyn. AL. HERRMANN. Schweiz. Verein von Gas- und Wasserfachmännern, Monatsbulletin **15**: 10, 262. October 1935. Coli strains are sensitive to silver and are quickly killed in katadyn water with 50 γ per liter of silver. Sterilizing action of silver ion is influenced by organic and inorganic matter and is entirely prevented by sodium chloride and albumen. Gram-positive germs, such as *Micrococcus roseus*, are more resistant; while spore-forming bacteria, such as *Bac. subtilis* and *Bac. mesentericus*, showed no evidences of growth inhibition during time of observation. Effect of katadyn on yeast is dependent upon character of natural or artificial substrate; fruit juice, for example, cannot be sterilized by usual silver concentrations. Oligodynamic principle is unsuitable for sure sterilization.—*Manz.*

Some Notes on Boiler Water J. BIERT. Schweizer Archiv für angewandte Wissenschaft und Technik, **1**: 6, 107. June 1935. Detailed review of current literature on boiler water conditioning is given. Author's opinion is that use of alkali as corrosion inhibitor is only justified with strong electrolytes present as chlorides, the alkali concentration being regulated according to chloride concentration. If aggressivity is caused by gases, complete removal of oxygen is necessary. High-pressure boiler feed with pure condensate and distilled water has repeatedly proved itself free from corrosiveness.—*Manz.*

The Significance of Ammonia for the Chlorine-Binding Power of Water. V. pH and Preammonisation. M. L. KOSCHKIN. Zeitschrift für Hygiene und

Infektionskrankheiten, **117**: 2, 182, July 1935. Chlorine-binding power of water is related to pH; with pH increasing from 2.8 to 8.2, chlorine-binding power dropped from 0.54 to 0.35 p.p.m., decrease being especially important in the range from 6.4 to 7.4. After preammoniation, a greater drop, to 0.14, was observed. Chlorophenolic taste and odor may not be prevented at low pH; after preammoniation, only at pH over 7.0. Bactericidal action of chlorine diminishes with increasing pH, with or without preammoniation; at pH 8.2, lack of sterilization after 2 hours contact is remarkable.—*Manz.*

A Hitherto Unknown Direct Cause of Corrosion in Hot Water Systems. L. W. Haase and G. Gad. Gesundheitsingenieur, **58**: 34, 526, August 1935. Attention was first arrested by the observation that after passing through copper heating coils, even at a very high rate of flow, complete removal of oxygen at 40°C. took place. New hot water systems with copper heating coils show first corrosion of galvanized iron service pipe close to copper installation, more distant pipes being later affected. Internal coating was of bright brownish color and contained small quantities of iron and lime; as much as 500 to 600 grams per square meter of zinc was found, so that poor coating could not have been responsible. Copper in amounts from 0.7 to 11.8 grams per square meter was also present and is considered as direct cause of this kind of corrosion. It is assumed that hot water, according to its chemical composition, dissolves copper in minute amounts, not detectable by usual water analysis, which are precipitated by first metallic surface with which they come into contact. Where metallic copper has been precipitated, corrosion proceeds, due to effect of local electrolytic cell copper/zinc and later copper/iron. In order to control this type of pipe corrosion, tinning of copper installation and pretreatment of water are recommended. As a practical means of prevention, phosphates, which after a short time build up a protective coating, have proven reliable.—*Manz.*

On Viability and Destruction of Germs in Drinking Water Containing Humic Matter. F. Sartorius and G. Wever. Gesundheitsingenieur, **58**: 29, 459, July 1935. Results of comparative bacteriological examination of water samples, with and without addition of humic acid, or peat extract, showed that the steady decrease of plate count observable with pure water is delayed in presence of humic matter. With *B. Coli*, after a lag of accommodation, increase is possible. With *B. typhosum*, distinct retardation of death rate was noticeable. It is concluded that humic matter may increase viability of organisms which may afterwards find their way into water supplies. It seems necessary, therefore, to remove humic matter from small supplies, which is readily accomplished with powdered activated carbon.—*Manz.*

On Lead in Drinking Water. F. Weyrauch and H. Müller. Zeitschrift für Hygiene und Infektionskrankheiten, **117**: 2, 196, July 1935. Study was undertaken in relation to origin of lead content of human bones. Analyses of 21 water samples not suspected of containing lead, from German cities free from lead disease, showed from 0.01 to 1.6 p.p.m. Pb after 12 hours standing in the pipes, and from 0.01 to 0.16 p.p.m. at midday, after considerable consump-

tion had taken place. As traces of lead found are considerable, especially after standing over-night; it is clear that appreciable carbonate hardness does not absolutely prevent plumbosolvency. After abundant run-off, samples proved practically lead-free; yet it must be supposed that quite often water that has stood in prolonged contact with lead pipe is imbibed. Ingestion of these small amounts of lead in drinking water may account for lead content of all human bones. Attention is called to the fact that lead is not in true solution: for, after filtration through asbestos, lead content dropped from 1.7 to 0.03 p.p.m.; and asbestos does not retain lead in true solution.—*Manz.*

NEW BOOKS

Metropolitan Water Board (London) 29th Chemical and Bacteriological Report for year ending December 31st, 1934. C. H. H. Harold. 83 pp. P. S. King & Son, Great Smith St., Westminster, London. 10 shillings and sixpence. Average daily consumption was 264.3 million gallons (in 1933 it was 295.1). Number of enteric fever cases notified was 109, decrease from 4,292 in 1900. During year 15,134 samples of water were examined bacteriologically and 4,566 chemically, and in addition 6,363 samples were examined by tasting. 88.3 per cent of samples of water going into supply were negative to *B. Coli* in 100 cc. Drought of 1934 lasted until December; but provision of huge storage systems, and public response to economy appeals were main factors in maintaining adequate supply. Analytical tables are given to show suitability of river Thames water for potable supply and prove that there is no serious deterioration of water as it passes downstream even in drought. Chloramine is still applied as bulk treatment with ammonium sulphate, followed by chlorine; but series of experiments were made using gaseous ammonia and ammonium chloride, with chlorine, conclusion being that most dependable treatment of all supplies and under all conditions is by preformed chloramines and preferably employing gaseous reagents. Experiments on extension of chloramine treatment to algae control were made, using copper sulphate, to find most effective proportions of chlorine, ammonia, and copper sulphate to employ on large scale. Most toxic results were with concentrations of components approximating to stoichiometric relation between chlorine and cuprammonium to which name "cuprichloramine" was given. This treatment was used throughout summer on New River and conclusion reached was that presence of ammonia, copper, and chlorine in water in appropriate proportions was conducive to best germicidal and algicidal results. These "cuprichloramine" investigations involved considerable work on estimation of small amounts of copper in water. Callan and Henderson's method (sodium diethyl-dithiocarbamate) was only satisfactory for clear waters and has been abandoned for rubeanic acid (dithio-oxamide) method. Experiments on ozone treatment were continued, conclusion being that ozone properly applied is effective sterilant, provided that at all times absence of breaches in filter barrier can be guaranteed; but that chloramine offers all round security under every possible condition and is "Safety First" of waterworks practice. For estimation of small amounts of nitrite in turbid waters, distillation method was tested and amounts up to 0.001 of nitrous nitrogen per 100,000 were recovered

almost completely. Griess-Ilosvay reagent is used and is satisfactory even in presence of free chlorine. Residual chlorine is determined by *ortho*-tolidine using dichromate and copper sulphate standards of Ellms and Hauser, and Hellige comparator gives results therewith agreeing with thiosulphate titration.—*W. G. Carey.*

Journal Southeastern Section, American Water Works Association. Proceedings, Seventh Annual Meeting, Birmingham, Ala. 1935. Publication Office, 522 Massey Bldg., Birmingham. 6 x 9 inches. 98 pp. **The Romance of Water.** R. E. Findlay. 5–8. Facts of popular interest about water and its relation to man. **The American Water Works Association and the Recovery Program.** Harry E. Jordan. 9–19. Public works construction, old age security, and other projects of the New Deal offer opportunities for co-operation of the water works fraternity. **Some Aspects of Interstate Carrier Water Supply Certification.** H. N. Old. 20–29. Of 2,105 supplies reported upon during 1933, 80 percent were certified favorably, 18.5 percent provisionally, and 1.5 percent were prohibited. Major undesirable features which resulted in provisional certification were failure to meet bacteriological standards, unimproved and uncontrolled cross-connections, inadequate treatment processes, improper location, or construction, of protective equipment, lack of covering for distributing reservoirs, and deficient operating control. In respect to cross-connections, their agency in spread of amoebic dysentery is considered. Bibliography. **Endeavors in Public Health Protection by Supervision of Water Supplies.** G. H. Hazelhurst. 30–41. Administration and procedure by Alabama State Department of Public Health. **Maintenance and Operating Kinks in Water Works and Sewerage.** F. E. Stuart. 42–57. Brief notes on many varied methods and devices developed in operating practice. **Service to Municipalities Rendered by Underwriters' Organizations maintained by Stock Fire Insurance Companies.** H. D. Cutter, Jr. 58–62. Investigation of, and advice upon, fire defense facilities are rendered in the interest of reduction of fire losses. **The Business Administration of Water Works.** W. W. Mathews III. 63–68. Customer relations considered in respect to application for service, meter reading, billing consumer's accounts, collections and settlement of accounts, investigation and adjustment of complaints, and charity accounts. **Elements of Sewage Treatment.** Sherwood Vermilye. 69–78.—*R. L. McNamee.*

Tientsin Water Works Department. British Municipal Council Report for 1934. Tientsin Press, Ltd. During the year a total of 8431 feet of new mains were laid and 231 new service connections made. Comprehensive overhaul of all machinery, auxiliary plant and wells was made early in the year in readiness for the summer load. The report includes tabulations and charts which indicate satisfactory progress in the operation of the system.—*A. W. Blohm.*

Electrical Measuring Instruments. Broadside E. Leeds & Northrup Company, Philadelphia, Pa. So that users can readily select the instrument they need for measurements in laboratory plant or field, the entire L & N line for

research, teaching and testing has now been listed and briefly described in a compact broadside which serves as a useful catalog.—*A. W. Blohm.*

Drinking Water Supply on Board of Mailsteamers. H. B. G. BREVER (Manager Medical Service Steam Navigation Cy., "Rotterdamsche Lloyd, Rotterdam, Holland") AND C. P. MOM (Director of Laboratory for Technical Hygiene, Technical College, Bandoeng, Java), Weekblad "De Ingenieur", 1935, 43. Drinking water on board mailsteamers ought to fulfill the terms of hygienic reliability, not only on account of the necessity of good drinking water, but still more with regard to its use in the preparation of foods and drinks. To supply good drinking water the process of "self-purification" and the use of Charcoal-, Bühring- and Chamberland-filters have not turned out to be effective.

Not so long ago drinking water was loaded in harbors of good reputation, but without any bacteriological control. The intake took place by means of canvas-hoses, connected with the drinking water supply on the quay or on waterboats. The filling gaps of the watertanks were on the level of the deck and to gauge the water a sounding-rod was used, which was kept in the shipcarpenter's cabin. Later on (in 1928) the filling gaps were fixed up above the deck-level; by this arrangement polluting by the deck streaming water was out of question.

Yet the drinking water supply on board was not satisfactory and consequently complaints about gastro-enteritis (gastric catarrh) often rose.

After this, purifying by means of filters came into use. Different kind of filters were used: Sand-, Charcoal-, Bühring-, Berkefeld- or Chamberland-filters. As it appeared necessary to disinfect the whole water distributing system chlorinated water (caporit) was used, which method is much better than cleaning the pipes with steam.

This produced a chlorine-taste and to eliminate this a Norit-filter (active carbon) was tried; by means of this apparatus the water could be de-chlorinated and at the same time freed from taste-matters. The results were satisfactory. The Norit-filters have proved to be of real benefit and perfectioning of this system seems to be valuable.

With regard to the question whether a decrease has been established in the frequency of acute gastro-enteritis among the crew and the passengers, it may be noted that after the installation of active carbon filters (1931) such outbreaks did not occur, while from 1928 to 1931 on three ships of the Rotterdamsche Lloyd three outbreaks of gastro-enteritis and one of para-typhoid B (in total 300 cases of illness) occurred.